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USE OF STORM-WATER BASINS FOR ARTIFICIAL RECHARGE WITH RECLAIMED WATER, NASSAU COUNTY, LONG ISLAND, NEW YORK — A HYDRAULIC FEASIBILITY STUDY

By

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CONVERSION FACTORS AND ABBREVIATIONS

Inch-pound Units	Multiply by	To obtain SI[1] Units
	Length	
inch (in)	25.4	millimeter (mm)
foot (ft)	.3947	meter (m)
mile	1.6093	kilometer
	Area	
square foot (ft^2)	.092903	square meter (m^2)
acre	.4047	hectometer (hm^2)
	Volume	
cubic foot (ft^3)	.02832	cubic meter (m^3)
	Flow	
feet per hour (ft/h)	.3048	meters per hour (m/h)
gallons per minute (gal/min)	.06309	liters per second (L/s)
million gallons per day per square foot [(Mgal/d)ft^2]	.04381	$meter^3$ per second (m^3/s)

1/ International system of units

USE OF STORM-WATER BASINS FOR ARTIFICIAL RECHARGE WITH RECLAIMED WATER, NASSAU COUNTY, LONG ISLAND, NEW YORK-- A HYDRAULIC FEASIBILITY STUDY

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ABSTRACT

A survey of 205 storm-water basins in Nassau County shows that 14 of the 50 largest basins would be suitable for dual infiltration of reclaimed water and storm runoff. Each basin would be divided by an earthen partition so that half would be used for storm-water retention, the other half for reclaimed water.

The 14 basins together could accommodate 38.6 million gallons per day of reclaimed water at infiltration rates as high as 0.28 feet per hour, which is approximately 20 percent of the basins' theoretical maximum infiltration capacity. However, the basins cannot accommodate the amounts needed to offset the ground-water deficit predicted for the 1990's in Nassau County.

The tendency of certain basins to overflow during large-magnitude storms could be reduced by enlarging the basins to accommodate excess runoff, lowering the height of the dividing partition, diverting excess runoff to another basin, or temporarily halting inflow of reclaimed water.

The regional water-table rise that would result from artificial recharge at the 14 selected basins would partly offset the decline in the water table and in streamflow caused by sewering in parts of Nassau and Suffolk Counties.

INTRODUCTION

Ground water is the sole source of freshwater for the 2.7 million residents of Nassau and Suffolk Counties on Long Island, N.Y. Under natural conditions, the ground-water reservoir is recharged only by local precipitation. Rapid population growth and urbanization of the island have increased demands for ground water, and the resulting increase in waste-water discharged through cesspool and septic-tanks to the ground threatens the quality of the ground water.

Present and planned water-management programs on Long Island call for the collection and treatment of wastewater and the elimination of septic tanks and cesspools to prevent contamination of the ground-water reservoir. However, the disposal of treated wastewater to Long Island's coastal waters will remove from the hydrologic system water that might otherwise have been returned to the ground-water reservoir. This is expected to significantly reduce the island's water supply. From projections of population growth, water use, and water consumption, Nassau County is expected to experience a water deficiency (the amount by which withdrawal exceeds recharge) of 92 Mgal/d by 1990 and 177 Mgal/d by 2020 (Greeley and Hansen, 1971).

To supply water in the future, Nassau and Suffolk Counties are studying several techniques, one of which is artificial recharge--returning reclaimed water, or highly treated wastewater, to the ground-water system (Cohen and others, 1968). One method of artificial recharge being investigated on Long Island is the use of storm-water basins.

About 650 storm-water basins in Nassau County were being used in 1978 for disposal of storm runoff. Because the surficial geologic formations on Long Island generally have high permeability, most basins dispose of storm runoff rapidly and empty within a few hours after a rainfall. As a result, most basins are empty most of the time and are therefore potential sites for artificial recharge, provided that (1) high infiltration rates can be maintained; (2) impoundment of reclaimed water would not interfere with a basin's primary function of storm-runoff disposal; and (3) the unsaturated deposits underlying the basins can accept the additional amounts of reclaimed water.

Purpose and Scope of Study

This study was done to determine which storm-water basins in southern Nassau County could most effectively dispose of reclaimed water and to predict and describe the combined hydraulic effects of storm runoff and reclaimed-water infiltration on the hydrologic system of Long Island. Specific objectives were to:

1. analyze the ability of selected storm-water basins to dispose of storm runoff and reclaimed water simultaneously without overflowing;

2. predict the net rise that such recharge would have on the regional water-table altitude in Nassau County and parts of neighboring Queens and Suffolk Counties;

3. predict the size and extent of water-table mounds at the recharge sites and the effect of these mounds on the quantity of reclaimed water that can be infiltrated; and

4. predict increases in streamflow that would result from infiltration of reclaimed water.

The first objective was studied with a mathematical model, the second and fourth by a regional electric-analog model, and the third by a digital model. This report describes the methods by which the above evaluations were made. Although the effects of water quality on the infiltration capacity of basins is mentioned, the chemical quality of reclaimed water and the change in ground-water quality resulting from artificial recharge are beyond the scope of this study.

Acknowledgments

This investigation was made in cooperation with the Nassau County Department of Public Works. Thanks are extended to the many individuals who contributed data or assisted in their compilation and analysis. Especially helpful in providing detailed design information on the many storm-water basins in Nassau County were Jack Murray, Senior Civil Engineer, and Jerry Becker, Drainage Design Section, both of the Nassau County Department of Public Works. Special thanks are also given to Francis J. Flood, Superintendent of Operations and Maintenance, Nassau County Department of Public Works, for his advice and assistance.

Location and Extent of Study Area

This study was restricted to basins south of the ground-water divide and north of Hempstead Turnpike in Nassau County. Only basins owned and maintained by Nassau County were investigated.

Location of the study area in Nassau County, the present (1976) ground-water divide, and major geographic features of Long Island are shown in figure 1.

Description of Storm-Water Basins in Nassau County

Storm-water basins on Long Island are unlined, open pits of various shapes and sizes excavated in moderately to highly permeable sand and gravel deposits of glacial origin. The basins dispose principally of storm runoff from highways and from residential, industrial, and commercial areas. The areas of individual basins in Nassau County range from about 0.1 acre to about 14 acres and average about 2 acres. Most basins extend 10 to 15 ft below land surface, but some are as shallow as 5 ft. In Nassau County in 1969, 557 county basins and 69 State Highway and State Parkway basins added approximately 20 Mgal/d of storm runoff to the ground-water reservoir (Seaburn and Aronson, 1974).

Most basins are equipped with some type of safety structure whereby overflow is carried through pipes, flumes, or street gutters to another basin or to a nearby stream.

The floors of many storm-water basins in Nassau County have two or more levels. The lower level is at the invert (bottom) of the inflow pipes and acts as a settling pool to collect inflowing sediment and trash. The upper, or bench, level is commonly 2 ft above the lower level and serves as a platform for maintenance work when the lower level is flooded. The bench level allows more rapid infiltration of water into the ground inasmuch as it remains comparatively free of sediment and debris.

Design criteria for storm-water basins in Nassau County in the last 40 years have evolved mostly on a trial-and-error basis. In Nassau County, the required basin volume below overflow elevation is estimated by multiplying (1) the volume of water equivalent to 5 inches of rainfall on the total drainage area surrounding the basin by (2) a runoff coefficient (ratio of runoff to precipitation) that generally ranges from 30 to 90 percent. A runoff coefficient of 40 percent is used in most residential areas, whereas a coefficient as high as 90 percent is used in industrial areas, where the amount of impervious surface area is greater.

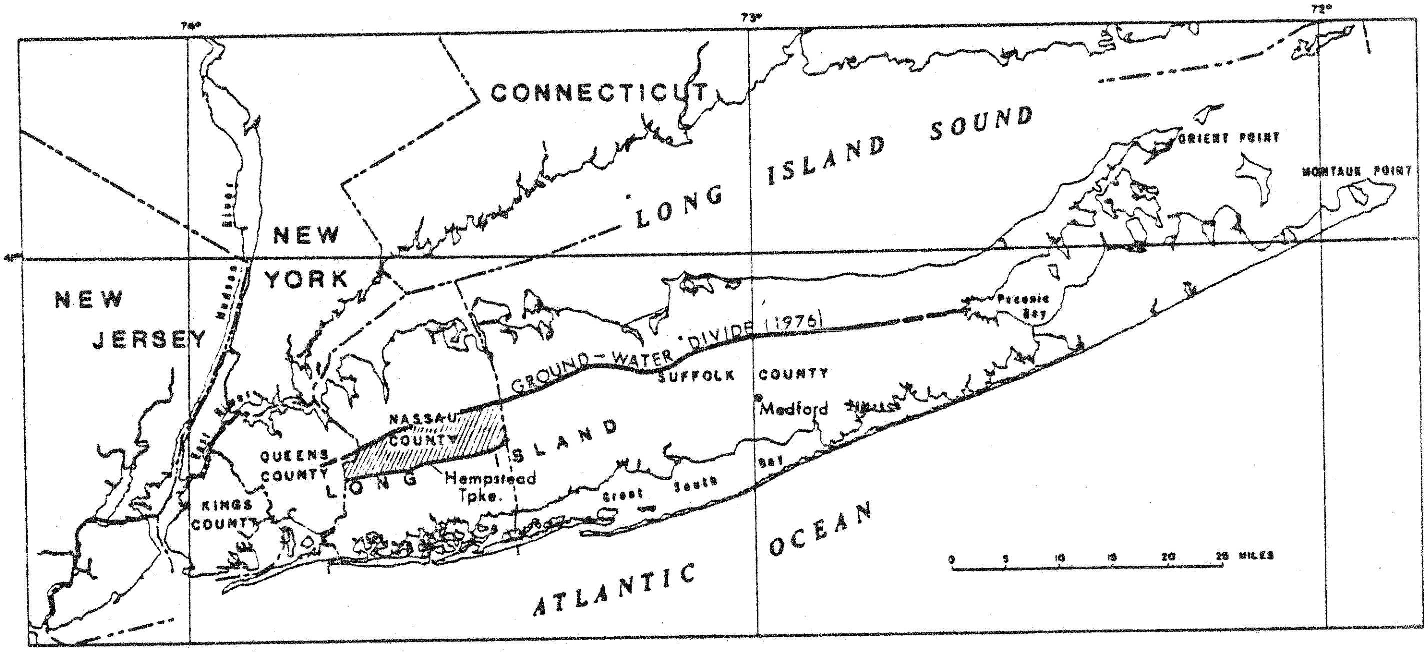

Figure 1.--Location and general geographic features of Long Island, N.Y., with study area (shaded) and approximate position of ground-water divide in 1976.

The depth and floor areas of a basin are calculated to contain the entire volume of inflow during the design storm. Infiltration of impounded water through the floor and sides of the basin during storm inflow is omitted from the calculation to provide a safety margin.

Hydrogeology

The hydrogeology of Long Island has been described in earlier reports, including those of Cohen, Franke and Foxworthy (1968) and McClymonds and Franke (1972). The study area has been described in detail by Perlmutter and Geraghty (1963); Pluhowski and Kantrowitz (1964); and Ku, Vecchioli, and Cerrillo (1975).

Long Island is underlain by a wedge-shaped mass of unconsolidated sedimentary deposits that overlie consolidated bedrock. The bedrock dips southeast from near sea level in northwest Queens County to about 2,000 ft below sea level in parts of Suffolk County's south shore. The lower unconsolidated deposits are of Cretaceous age and dip southeast also. They are mantled nearly everywhere on Long Island by Quaternary glacial deposits. A generalized description of the hydrogeologic system underlying the study area in central Nassau County is given in table 1.

PROPOSED ARTIFICIAL-RECHARGE SYSTEMS

To ensure an adequate water supply in the future, Nassau County has instituted a Comprehensive Master Plan Development Study to be used as a planning guide for the next 50 years. That study, which (1) provides estimates of immediate and long-term land use and development and population shifts, and (2) assesses water-management technology, also evaluates the physical requirements for implementation of a water-management program. These requirements include water sources, waste-treatment facilities, water-transmission lines, pumping stations, and storage facilities (Greeley and Hansen, 1971).

Part of the Comprehensive Master Plan Development Study involves recycling reclaimed water. One phase of the study--the Advanced Waste-Water Treatment Plan--is a short-term demonstration project to assess the performance and reliability of a proposed 5-Mgal/d wastewater-treatment and ground-water-recharge operation. Reclaimed water would be supplied by the Cedar Creek Pollution Control Plant, now (1978) under construction, and would be pumped 7 miles to recharge facilities at the Meadowbrook Artificial-Recharge site. Locations of the Cedar Creek Plant, the Meadowbrook site, and the transmission main are shown in figure 2. Nassau County storm-water basin 62, now used for disposal of storm runoff, would be used for emergency disposal of excess reclaimed water. If reclaimed water should become available in sufficient amounts, studies may be made at basin 62 to evaluate the effectiveness of using storm-water basins for artificial recharge.

Another phase of the Comprehensive Master Plan Development study--the Recycle Supplemental Supply Plan--is a long-range program to return as much as 94 Mgal/d of reclaimed water to the ground-water reservoir by the year 1990 through storm-water basins (Greeley and Hansen, 1971). Reclaimed water would be supplied by the Bay Park Sewage Treatment Plant (fig. 2), and the Cedar Creek Pollution Control Plant and would be pumped to basins through transmission mains. The success of this program will be dependent, in part, on results achieved by the Advanced Wastewater Treatment Plant described previously.

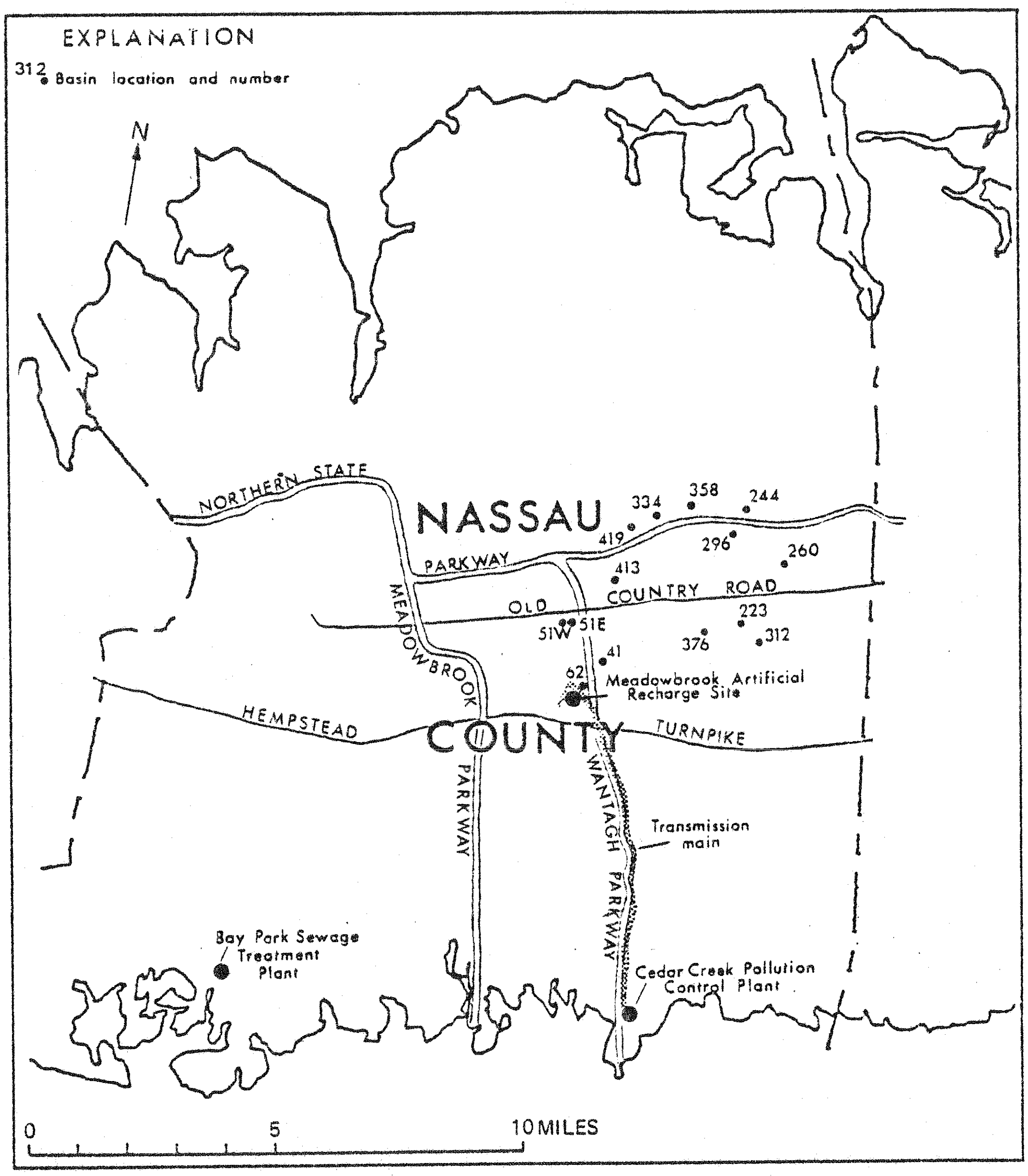

Figure 2.--Locations of the 14 selected storm-water basins, Bay Park Sewage Treatment Plant, Cedar Park Pollution Control Plant, Meadowbrook Artificial-Recharge site, and proposed transmission main.

Table 1.--Generalized description of hydrogeologic units that underlie southern Nassau County, N.Y. 1/

Hydrogeologic unit	Approximate range in thickness (feet)	Character	Water-bearing properties
Holocene deposits artificial fill, salt-marsh deposits, and shore deposits	0-40	Sand, gravel, clay, silt, organic mud, peat, loam, and shells; gray and brown.	Sandy beds of moderate to high permeability beneath barrier beaches, locally yield fresh or salty water from shallow depths. Clayey and silty beds beneath bays retard salt-water encroachment and confine underlying aquifers.
Upper glacial aquifer	30-300	Outwash consisting mainly of brown fine to coarse sand and gravel, stratified. Interbedded with "20-foot" clay.	Sand and gravel part of outwash highly permeable; yields of individual wells are as much as 1,700 gal/min. Specific capacities of wells as much as 109 gal/min per foot of drawdown. Water fresh except near shorelines.
"20-foot" clay	0-40	Clay and silt, gray and grayish-green; some lenses of sand and gravel. Contains shells, foraminifer, and peat. Altitude of top of unit about 20 ft below mean sea level. Interbedded with outwash in southern part of area.	Relatively impermeable confining unit. Retards salt-water encroachment in shallow depths.
Gardiners clay	0-40	Clay and silt, grayish-green; some lenses of sand and gravel. Contains lignitic material, shells, glauconite, foraminifers, and diatoms. Interglacial deposit. Altitude of surface about 50 ft or more below mean sea level.	Relatively impermeable confining layer above Jameco aquifer. Locally contains moderately to highly permeable sand and gravel lenses.
Jameco aquifer	0-300	Sand, fine to coarse, dark-gray and brown; gravel. Contains some thin beds of silt and clay. Probably older glacial outwash.	Highly permeable. Yields as much as 1,300 gal/min to individual wells. Specific capacities as high as 135 gal/min per foot of drawdown. Contains water under artesian pressure. Water commonly has high iron content and is salty near shoreline.

Table 1.--Generalized description of hydrogeologic units that underlie southern Nassau County, N.Y. (Continued)

Hydrogeologic unit	Approximate range in thickness (feet)	Character	Water-bearing properties
Magothy aquifer	200-900	Sand, fine to medium gray; interfingered with lenses of coarse sand, sandy clay, silt, and solid clay. Generally contains gravel in bottom 50 to 100 ft. Lignite and pyrite abundant.	Slightly to highly permeable. Principal source of public-supply water in Nassau County. Individual wells yield as much as 2,200 gal/min. Specific capacities as high as 80 gal/min per foot of drawdown. Water mainly under artesian pressure; some wells in southern part of area flow. Water generally is of excellent quality except where contaminated by salty water in southwestern part of area or by dissolved constituents associated with activities of man.
Raritan clay	100-300	Clay, gray, white, and some red and purple; mainly in solid silty and rarely sandy lenses. Some lenses of sand and gravel. Lignite and pyrite abundant.	Relatively impermeable confining unit. Local lenses and layers of sand and gravel, moderate to high permeability.
Lloyd aquifer	150-300	Sand, fine to coarse, gray and white, and gravel; some lenses of solid and sandy clay, and clayey sand. Thin beds of lignite locally.	Moderately permeable. Yields as much as 2,000 gal/min to individual wells. Specific capacities as high as 44 gal/min per foot of drawdown. Water under artesian pressure; some wells flow. Water of good quality except for high iron content.
Bedrock	--	Crystalline metamorphic and igneous rocks. Soft, clayey weathered zone at top, as thick as 50 ft.	Virtually impermeable. Locally contains water along joints and fault zones.

1/ Adapted from Ku, H. F. H., Vecchioli, John, and Cerrillo, L. A., 1975, Hydrogeology along the proposed barrier-recharge-well alignment in southern Nassau County, Long Island, New York: U.S. Geological Survey Hydrologic Investigation Atlas HA-502.

The conditions, materials, and specifications required for these water-reclamation and recharge plans were used as a guide in selecting storm-water basins suitable for infiltration of reclaimed water.

SELECTION AND DESCRIPTION OF TEST BASINS

A survey of 205 recharge basins south of the ground-water divide and north of Hempstead Turnpike (fig. 1) was made to determine which basins would be best suited for infiltration of reclaimed water (Aronson, 1976a). Of these, 150 of the largest basins, as determined from basin-design drawings, were examined in the field to verify storage capacity and maximum infiltration area and to rank the basins according to size. The 50 largest basins were then evaluated for their suitability as potential reclaimed-water infiltration sites. Among the factors considered were (1) proximity to planned transmission mains, (2) tendency of the basin to retain water too long after a rainfall, (3) permeability of underlying geologic units, and (4) adequate depth to the water table. Fourteen of the 50 largest basins met all criteria and were studied further as potential sites for infiltration of reclaimed water. The locations of the 14 selected storm-water basins are shown in figure 2; other pertinent information, in order of decreasing maximum infiltration areas, is given in table 2.

The ability of these basins to accept reclaimed water depends, in large part, on their infiltration characteristics and on the capacity of the unsaturated zone and the underlying aquifers to accept large volumes of reclaimed water. The ability of the 14 test basins to accommodate storm runoff and reclaimed water simultaneously without overflowing, and the local and regional impact of such recharge on the altitude of the water table, were investigated by mathematical, digital, and electric-analog models. The results of that investigation are described in the two sections that follow.

DESCRIPTION OF MODELS

Mathematical Model

The ability of storm-water basins to transmit reclaimed water and storm runoff without outflowing was evaluated by the finite-difference mathematical model described by Aronson and Prill (1977) and Aronson (1976b). This model simulates the maximum amount of water that would enter a basin as runoff from statistically derived, large-magnitude storms with or without simultaneous ponding of reclaimed water. Analyses of basin characteristics during dual recharge were used to determine (1) the quantity of water that will pond in the test basins during the most severe storms; (2) duration of runoff containment after rainfall has ceased; (3) minimum allowable infiltration rates; and (4) maximum allowable runoff coefficients.1/

1/ Runoff coefficient indicates, as a percentage, the ratio of runoff to total precipitation in a given area.

Table 2.--Physical and hydrogeologic data on the 14 test basins, Nassau County, N.Y.

Basin	Geographic location (Village)	Date of acquisition by county	Contributing drainage area (acres)	Storage capacity[1/] (ft^3)	Maximum infiltration area[2/] (ft^2)	Floor elevation (feet above sea level)	Depth to water table[3/] (ft)
62	East Meadow	1-29-52	419.9	6,400,000	440,000	79	13
51 West	East Meadow	12-07-42	239.4	1,738,000	387,200	96	20
334	Jericho	4-04-60	172.8	2,668,000	284,000	158	70
358	Jericho	4-04-60	212.4	1,940,000	278,400	163	75
51 East	East Meadow	12-07-42	273.5	2,090,000	256,000	97	21
413	Hicksville	5-12-58	396.3	2,740,000	230,000	111	25
376	Hicksville	10-05-56	92.0	1,228,000	151,100	116	40
419	Jericho	2-20-62	162.0	1,280,000	150,200	137	50
296	Plainview	11-04-57	117.4	1,342,000	138,200	164	77
260	Plainview	11-14-55	96.5	701,000	116,900	145	63
244	Syosset	10-17-55	104.8	738,000	114,500	178	92
223	Bethpage	7-12-54	94.4	688,000	93,200	126	45
312	Bethpage	12-22-55	81.2	630,000	82,500	120	46
41	Levittown	6-11-52	106.0	653,000	65,200	92	30

1/ Storage capacity is the volume of water the basin can contain without overflowing.
2/ Maximum infiltration area is the horizontal projected plan area at the overflow elevation.
3/ Depths measured from basin floor based on 1974 water-table altitude.

Digital Model

Any model designed to predict the development of ground-water mounds beneath recharge basins on Long Island must be able to simulate unconfined, nonhomogeneous, and anisotropic aquifer conditions and to reflect the variability of aquifer characteristics on Long Island (as shown in table 1). A digital model was used in this study because no mathematical solutions can fully predict the response of the water table to basin recharge in anisotropic aquifers.

The model used is a "Galerkin finite-element" model that solves the equation for transient, two-dimensional (vertical and horizontal), radially symmetric saturated flow. It can incorporate the different hydraulic properties of Long Island's aquifers as needed to give an accurate prediction of water-table mounding beneath infiltration sites. It can also predict the resulting increases in hydraulic head through time at given depths beneath, and radial distances from, recharge sites and can thus predict head buildup throughout the entire aquifer system of Long Island.

Analog Model

A three-dimensional electric-analog model that simulates the Long Island ground-water system to the base of the Magothy aquifer was used to simulate the rises in altitude of the regional water-table and increases in the amount of local streamflow that supplemental recharge would cause (Getzen, 1977). All of Long Island except the eastern forks is represented in this model. The two major aquifers and associated confining beds are modeled in five layers, of which the upper glacial aquifer is represented by two, the Magothy aquifer by three. (See table 1.) Water-level changes can be simulated only on a regional basis because the distance represented between nodes is relatively large (6,000 ft); therefore, changes in water-table altitude that affect a half-node interval (3,000 ft) or less are neglected in this model.

ABILITY OF SELECTED BASINS TO ACCEPT

RECLAIMED WATER AND STORM RUNOFF

Basin Design and Operation

Aronson (1976b) analyzed several methods of applying reclaimed water and storm runoff to basins originally designed solely for infiltration of storm runoff. The "partitioned-basin" method of water application was found to be the most advantageous. In this method, a storm-water basin is partitioned by an earthen wall to form two separate but equal infiltration areas. Partition height is approximately 5 ft but could be lower for certain conditions. (Modification of partition height to suit operating conditions is discussed later in this report.) In a partitioned basin, reclaimed water is pumped into

half the basin and the other half is used for infiltration of storm runoff. After a predetermined infiltration period, the function of each half is reversed, and the side previously charged with reclaimed water is rested to permit maintenance of the basin floor and restoration of infiltration rate. A plan view and cross section of this configuration is depicted in fig. 3. McGauhey and Krone (1967) demonstrated that infiltration rate declines during continuous application of water regardless of water quality. The reduction of infiltration rate could be minimized, however, by use of regular drying periods between water applications. During such rest periods, maintenance procedures such as tilling or scraping the basin floor and removing debris could be done in the basin half last used for infiltration of reclaimed water.

Method of Basin-Performance Analysis

Basic Equations

Impoundment of reclaimed water in a basin will reduce the storage and infiltration area available for storm runoff; that is, the area occupied by reclaimed water will be unavailable for storm runoff and will develop a lower infiltration rate. A reduction in either or both of these factors may result in an inadequate storage capacity for the combined volumes of runoff and reclaimed water during large-magnitude storms. Therefore, these factors must be considered in any evaluation of a partitioned basin's ability to accept both runoff and reclaimed water.

Changes in volume of stored water in a partitioned basin during dual recharge can be defined through a modification of a procedure for determining changes in water storage in a basin during inflow of storm runoff (Aronson and Prill, 1977) by the continuity equation

$$S_t = W + \sum_{i=1}^{n} (V_i - I_i) \qquad (1)$$

where:

- S_t is the volume of stored water at a specified time after the start of precipitation, in cubic feet;
- W is the volume of reclaimed water stored in the basin, in cubic feet;
- i is the number of the time interval since the start of precipitation;
- V_i is the volume of storm runoff added to basin during the i^{th} time interval, in cubic feet;
- I_i is the volume of water that infiltrates through the basin during the i^{th} time interval, in cubic feet;
- n is the number of time intervals used to determine S_t.

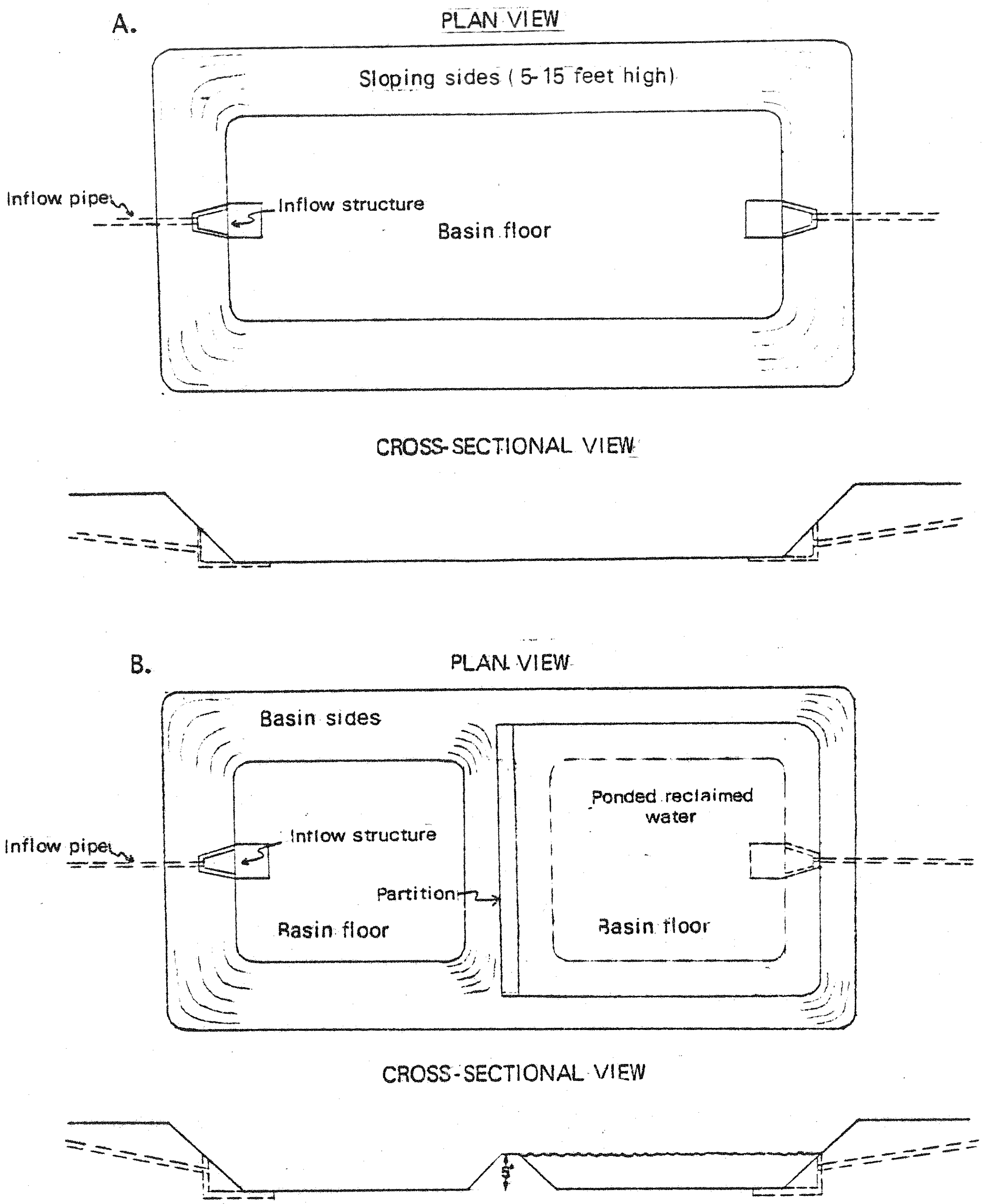

Figure 3.--Plan view and cross section of storm-water basin. A, idealized, unmodified basin; B, same basin with partition to allow ponding of reclaimed water.

The procedure requires that duration of both inflow to the basin and infiltration within the basin be divided into sufficiently short time intervals that the net change in amount of stored water can be considered equivalent to the sum of the incremental differences between volumes of inflow and infiltration. Because most storm-water basins on Long Island usually empty within one or two days after a large storm, time intervals ranging from 5 minutes to 12 hours were used.

The computational procedure for solving equation 1 requires that cumulative storage be determined at the start of each time interval. Once this is done, the infiltration area at the start of that time interval is determined from graphs that relate basin stage, infiltration area, and storage volume. An example of such a graph is given in figure 4. If the infiltration area is assumed to remain constant throughout the interval, equation 1 can be expressed as

$$S_x = S_o + V_i - (A_o \cdot R \cdot t_i) \tag{2}$$

where:

S_x is a preliminary estimate of the volume of water in storage at the end of the i^{th} interval, in cubic feet;

S_o is the volume of water in storage at the beginning of the i^{th} interval, in cubic feet;

A_o is the infiltration area for S_o, in square feet;

R is the infiltration rate for A_o, in feet per hour;

t_i is the duration of the i^{th} interval, in hours.

S_x is a preliminary estimate of S_t in equation 1. During periods of rising stage, the value of S_x is an overestimate of water in storage because the infiltration area and, correspondingly, the volume of water infiltrating are constantly increasing. Similarly, during periods of declining stage, the value of S_x is an underestimate of maximum storage (S_t) because the infiltration area is decreasing. Thus, to calculate maximum storage (S_t), one must first calculate the average infiltration area (A_i) during the given time interval. This can be computed from the equation

$$A_i = \frac{A_o + A_x}{2} \tag{3}$$

where:

A_o is the infiltration area for S_o, in square feet, at the start of the time interval;

A_x is the infiltration area for S_x, in square feet, at the end of the time interval.

The procedure used to determine average infiltration area (A_i) during a selected time interval is based on a predictor-corrector scheme, where S_x is the predictor term, computed from area A_o at S_o, and the corrector term is $(A_o + A_x)/2$, where A_x is computed from the predictor term. Subtracting the average infiltration area (A_i) from the volume of water in storage at the start of the time interval (S_o) and adding the volume of stored reclaimed water (W) and the volume of storm runoff added to storage during the interval (V_i) gives the volume of water in the basin at the end of the time interval (S_t). The process is repeated for each time interval within the storm

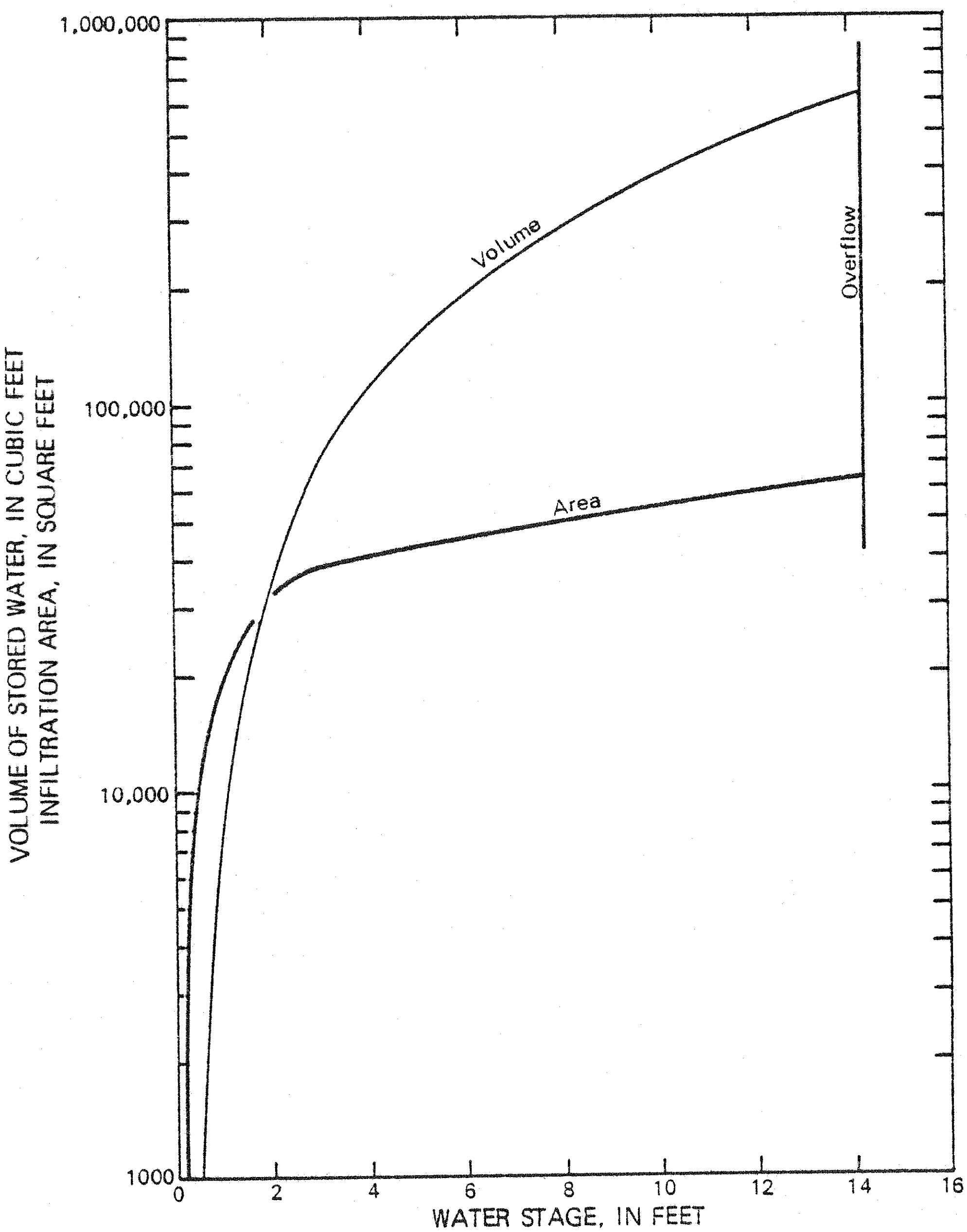

Figure 4.--Relationship between infiltration area, volume of stored water, and water stage at basin 41.

duration. Therefore, the volume of water stored in the basin at the end of the storm is equivalent to the sum of incremental changes in water storage during the storm $\left[\sum_{i=1}^{n} (V_i - I_i) \text{ in equation 1}\right]$

The length of time intervals (t_i) used to determine S_t is a compromise between accuracy and computational cost. As a general rule, the interval should be short enough that S_x, as computed by equation 2, is not appreciably larger than S_o. The short time intervals used for the computations in this report were made possible by minicomputer. These intervals range from 5 minutes to 12 hours.

The analyses of basin performance presented in this report require that certain factors be quantified, specifically (1) those that relate to the magnitude, duration, and intensity of a storm; (2) basin characteristics and dimensions, runoff coefficient of the basin drainage area, infiltration rate of the basin floor; and (3) method of water application. The significance of each of these factors in determining a basin's suitability for supplemental recharge is discussed in the following sections.

Storm characteristics--Large-magnitude storms provide the best evaluations of basin performance during disposal of storm runoff and reclaimed water because the basin's ability to dispose of runoff is under maximum stress at these times. Analyses of storm-frequency and storm-intensity data for the New York City area were made by Miller and Frederick (1969) to determine the return periods of rainstorms of given intensities and durations. Return period, also known as recurrence interval, or frequency, is the average number of years within which a storm of given magnitude is equaled or exceeded (Chow, 1964, sec. 8, p. 22). Thus, a 10-year storm is of lesser magnitude than a 100-year storm. In this report, storms having return periods of 10, 25, 50, and 100 years were used to evaluate basin performance during disposal of reclaimed water. Table 3 summarizes the relationship between storm frequency, storm duration, and precipitation intensity in the New York City area as determined by Miller and Frederick (1969). Each storm duration and intensity listed in table 3 was used in analyses of basin operation to determine the most critical combination for each basin.

Runoff coefficient--In much of Nassau and Suffolk Counties, precipitation on impervious surfaces such as roofs, driveways, sidewalks, paved streets, and, to a considerably lesser extent, runoff from unpaved land, drains into storm sewers that empty into storm-water basins. The ratio of the amount of runoff to the amount of rainfall is the runoff coefficient.

Studies by Seaburn (1970) and Seaburn and Aronson (1974) have shown that the percentage of total inflow to a basin from large-magnitude storms is nearly proportional to the percentage of impervious area within the basin's drainage area. However, the runoff coefficient used in the design of residential basins in the study area (40 percent) is considerably larger than the percentage of impervious area within the basins' drainage areas; this is to allow for storms of extraordinary intensity or frequency, or for storms that occur when the ground is frozen, during which time the runoff coefficient

may increase and the infiltration rate of the basin may decrease considerably. Therefore, the design runoff coefficient of 40 percent was used in this study to evaluate performance of the 14 basins during conditions of maximum stress.

Infiltration rate--The infiltration rate of a basin is the rate at which water percolates through the wetted area, measured in feet of water per hour. This is probably the most variable factor involved because it depends on many other factors, including water temperature, water stage, soil-moisture conditions, water quality, and length of flooding period. In analyses of basin performance, the infiltration rate during impoundment of storm runoff is probably best treated as a constant despite its variability because changes in infiltration rate at typical storm-water basins on Long Island during impoundment are generally small (Seaburn and Aronson, 1974; Prill and Aronson, 1978), and an average infiltration rate has been found sufficient to predict basin operation.

Table 3.--Average precipitation intensity during 10-, 25-, 50-, and 100-year storms in the New York City area

[Modified from Miller and Frederick, 1969]

Storm duration		Average precipitation intensity (inches per hour)			
Hours	Minutes	10-year storm	25-year storm	50-year storm	100-year storm
0	5	7.10	7.75	8.61	9.70
0	10	5.35	6.09	6.90	7.78
0	15	4.45	5.20	5.90	6.60
0	30	3.10	3.78	4.30	4.80
0	45	2.49	3.02	3.49	3.90
1	--	2.05	2.58	2.92	3.30
2	--	1.35	1.72	1.92	2.12
3	--	1.02	1.28	1.48	1.64
4	--	.83	1.03	1.17	1.32
8	--	.51	.62	.70	.78
10	--	.43	.51	.59	.65
12	--	.38	.45	.51	.57
14	--	.33	.40	.46	.50
18	--	.28	.33	.37	.41
24	--	.22	.26	.26	.33
36	--	.17	.19	.21	.24
48	--	.13	.15	.17	.19
60	--	.11	.13	.14	.16
72	--	.09	.11	.12	.14

An average infiltration rate of 1.0 ft/h was assumed for the basin walls above the level of ponded reclaimed water. This rate is an approximation of the average infiltration rate of several storm-water basins studied by Seaburn and Aronson (1974) and Prill and Aronson (1978); furthermore, studies of basin recharge with reclaimed water at Medford in Suffolk County (fig. 1) indicate that infiltration rates of at least 1.0 ft/h can be maintained for many days (R. C. Prill, oral commun., 1977). However, to assess the effect of a declining infiltration rate during ponding of reclaimed water for periods of several days, infiltration rates of less than 1.0 ft/h below the level of ponded water were also analyzed. One of the reduced infiltration rates analyzed was 0.28 ft/h [50 $(gal/d)/ft^2$]--a rate suggested by Greeley and Hansen (1971, p. 129) as a reasonable design criterion on the basis of observed infiltration rates during long-term infiltration of secondary-treated sewage at the Meadowbrook sewage-treatment plant at East Meadow in Nassau County (now the site of the Meadowbrook Artificial-Recharge site, fig. 2). Infiltration rates lower than 0.28 ft/h, resulting from prolonged ponding of reclaimed water, were also analyzed to predict basin performance under maximum-stress conditions even though very low rates are not expected if the basin floor is properly maintained during resting periods. When reduced infiltration rates are assigned to the impoundment area below the top of the basin partition, it is assumed that the infiltration rate of the basin half used for storm runoff has been reduced by prior recharge with reclaimed water.

Infiltration area--During inflow of storm runoff to a basin, the water level, and hence the infiltration area of impounded water, constantly increases. To calculate the average infiltration area (A_i) during the selected time interval (t_i), values A_o and A_x in equations 2 and 3 must be determined. The infiltration area for any given volume of water at the start and at the end of time period t_i can be established from the relationship between infiltration area and volume of stored water for that basin. The graph in figure 4 shows such a relationship for basin 41. This relationship can be used to determine the infiltration area for any volume of stored water. Graphs similar to that in figure 4 were made for each of the 14 test basins for use in computer analyses of basin performance during impoundment of stormwater and reclaimed water.

The maximum infiltration area of the 14 test basins totals 2,620,900 ft^2 or 60.2 acres. The relationship between storage and infiltration area in these basins indicates that the infiltration area available for reclaimed water ponded to a depth of 5 ft in half of each basin ranges from 32 to 46 percent of the maximum infiltration area of the basin (table 4); the remaining infiltration area--the other half of the basin and the walls of the entire basin above the ponding level of reclaimed water--is reserved for storm runoff. Thus, a total of 1,097,100 ft^2 or 25.2 acres would be available for infiltration of reclaimed water in the 14 test basins if the partitioned-basin method of water application were used.

Analysis of Basin Performance During Impoundment of Storm Runoff and Reclaimed Water

The methods and types of analyses used to evaluate the suitability of the 14 test basins for artificial recharge during large-magnitude storms are best explained with one of the test basins (basin 41) as an example. The basin is assumed to have infiltration rates of 0.28 ft/h below and 1.0 ft/h above the top of a 5-ft partition that divides it into two approximately equal halves. Reclaimed water is assumed to be ponded in half the basin to a depth of 5 ft, and partition thickness is assumed to be zero. The runoff coefficient is 40 percent. (Lag times between start of precipitation and inflow to the basin are not required for performance analysis.) The above factors were used to compute changes in water storage, from equation 3, for a series of idealized 10-year storms of different durations. An example of the results, in computer format, is given in table 5. The storms have durations of 5, 10, 15, 30, 45, 60, and 120 minutes and average precipitation intensities of 7.10, 5.35, 4.45, 3.10, 2.49, 2.05, and 1.35 in/h, respectively (from table 3). Precipitation intensity and runoff are assumed to be uniform during each storm; uniform runoff results in a progressive increase in water storage in the basin until a maximum is reached at the end of the storm. The nature of the increase at basin 41 is illustrated by a graph in figure 5, where storage values from table 5 are plotted for the 0.5-hour, 1-hour, and 2-hour storms.

Table 4.—Infiltration (wetted) areas of test basins when reclaimed water is ponded to a depth of 5 ft in half of each basin

[Areas are in square feet rounded to nearest 100]

Basin	Infiltration area of half the basin at 5-ft stage	Percentage of maximum infiltration area of basin1/
51 WEST	143,000	37
62	142,500	32
358	124,000	45
334	122,500	43
51 EAST	89,500	35
413	89,500	39
376	65,500	43
419	65,300	44
296	56,000	41
260	53,500	46
244	50,500	44
223	38,100	41
312	35,400	43
41	21,800	33
Total	1,097,100	

1/ Values rounded to nearest percent.

If computations such as those in table 5 are continued for 10-year storms of longer than 2-hour duration, storage values at the end of each storm can be used to generate a graph that shows maximum basin storage over a wide range of durations. Figure 6 plots the maximum quantity of water in storage at basin 41 at infiltration rates of 0.28 ft/h below and 1.0 ft/h above the partition for any 10-year storm with a duration of up to 72 hours. Figure 6 shows that the greatest stress would be from a storm of about 10-hour duration and that the maximum volume of water (storm runoff and reclaimed water) in storage from this storm would be 581,600 ft^3, which is 89 percent of the basin's capacity.

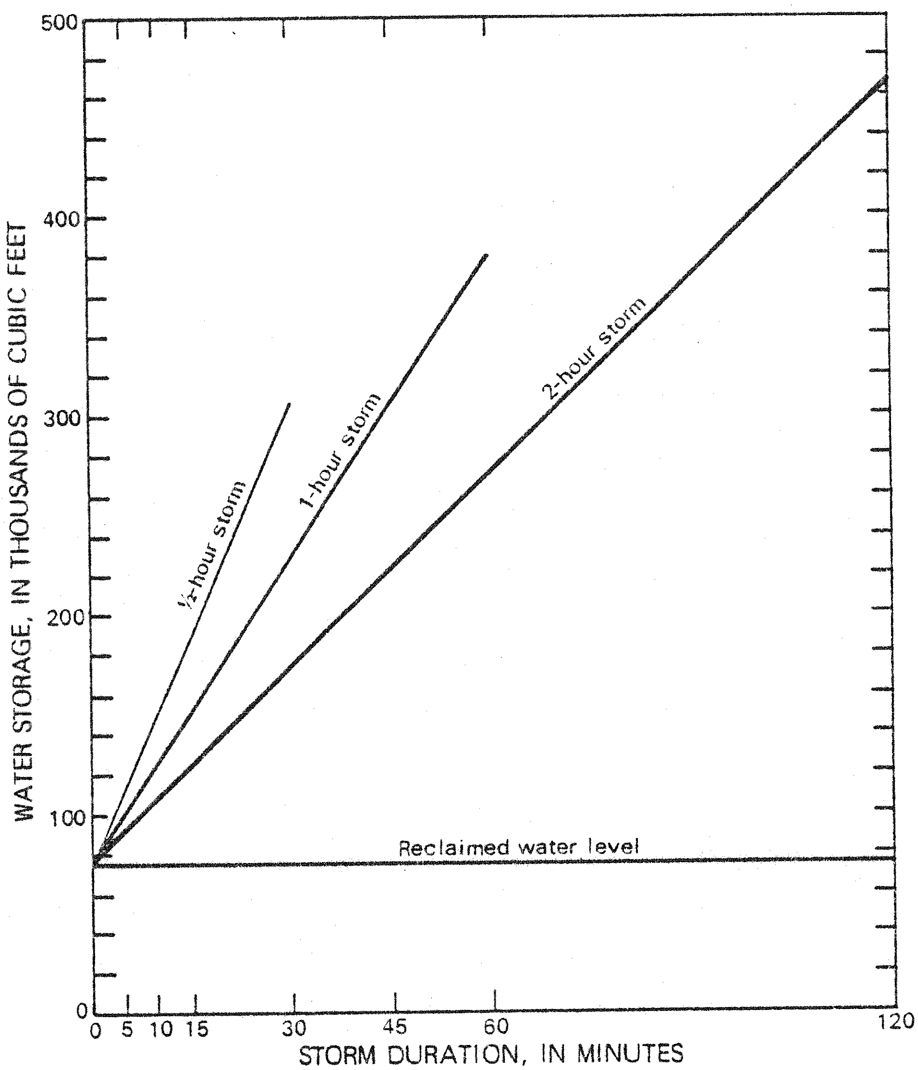

Figure 5.--Volume of water (stored runoff and reclaimed water) in basin 41 during 10-year storms of differing duration, at infiltration rates of 0.28 ft/h below and 1.0 ft/h above the 5-ft partition and a runoff coefficient of 40 percent.

Table 5.--Facsimile of computer printout listing calculated changes in water storage in basin 41 for 10-year storms of 5-minute to 2-hour durations, with reclaimed water ponded in half the basin to a depth of 5 ft, and a runoff coefficient of 40 percent

INFILTRATION RATE 0.28 FT/H BELOW AND 1.0 FT/H ABOVE 5-FT PARTITION
DRAINAGE AREA = 105.98 ACRES
RUNOFF COEFICIENT = 40 PERCENT
STORM RETURN PERIOD = 10 YEARS
VOLUME OF RECLAIMED WATER W = 75000 CUBIC FEET
INFILTRATION AREA OF RECLAIMED WATER AW = 21750 SQUARE FEET

TIME[1]	PREC INT[2]	CUM PREC[3]	CUM RUN[4]	RUN INCR[5]	SO[6]	2SO OR (SO+W)[7]	A2SO OR ASO+AW[8]	AO[9]	AVORT[10]
0	7.10	0	0	91047	0	0	0	0	0
5	7.10	.592	91047						
0	5.35	0	0	68606	0	0	0	0	0
5	5.35	.446	68606	68606	68357	136713	42755	21377	499
10	5.35	.892	137212						
0	4.45	0	0	57065	0	0	0	0	0
5	4.45	.371	57065	57065	56823	113646	41461	20730	484
10	4.45	.742	114130	57065	113316	188316	45329	23573	660
15	4.45	1.112	171195						
0	3.10	0	0	39753	0	0	0	0	0
5	3.10	.258	39753	39753	39522	79044	39563	19781	462
10	3.10	.517	79506	39753	78783	153783	43681	21931	523
15	3.10	.775	119259	119259	117936	192936	45549	23799	2035
30	3.10	1.550	238518						
0	2.49	0	0	31931	0	0	0	0	0
5	2.49	.207	31931	31931	31710	63420	37747	18873	440
10	2.49	.415	63861	31931	63175	126349	42173	21087	492
15	2.49	.622	95792	95792	94567	169567	44434	22684	1756
30	2.49	1.245	191584	95792	118041	236041	48894	27144	2871
45	2.49	1.867	287376						
0	2.05	0	0	26288	0	0	0	0	0
5	2.05	.171	26288	26288	26078	52156	35950	17975	419
10	2.05	.342	52577	26288	51918	103836	40932	20466	478
15	2.05	.512	78865	78865	77708	152708	43629	21879	1555
30	2.05	1.025	157730	78865	154589	229589	47051	25301	2410
45	2.05	1.537	236595	78865	230524	305524	51176	29426	3442
60	2.05	2.050	315460						
0	1.35	0	0	17312	0	0	0	0	0
5	1.35	.112	17312	17312	17127	34254	31544	15772	368
10	1.35	.225	34624	17312	34031	68063	38307	19154	447
15	1.35	.337	51935	51935	50882	101763	40824	20412	1429
30	1.35	.675	103871	51935	101184	176184	44750	23000	1835
45	1.35	1.012	155806	51935	151014	226014	46905	25155	2374
60	1.35	1.350	207742	207742	200239	275239	49575	27825	12165
120	1.35	2.700	415484						

1/ Time since start of precipitation, in minutes. Time intervals (t_i) are in multiples of 5 minutes. Last time interval in each set represents total storm duration.
2/ Precipitation intensity for selected storm duration, in inches per hour.
3/ Cumulative precipitation since start of storm, in inches.
4/ Cumulative volume of runoff since start of storm, in cubic feet.
5/ Incremental increase in runoff during specified time interval, in cubic feet (V_i in equations 1 and 2).
6/ Volume of runoff in basin at start of time interval, in cubic feet (S_o in equation 2).
7/ If $S_o \leq W$, S_o is doubled; if $S_o > W$, W is added to S_o.
8/ If $S_o \leq W$, infiltration area for doubled S_o is determined from volume/area curve (fig. 4) and is then halved to obtain infiltration area for S_o; if $S_o > W$, infiltration area for $S_o + W$ is determined from volume/area curve (fig. 4), and the infiltration area for W is subtracted to obtain the infiltration area for S_o.
9/ The net infiltration area for storm runoff, in square feet (A_o in equation 2).
10/ First estimate of volume of water infiltrated during t_i, in cubic feet. If $A_o \leq AW$, infiltration rate R = 0.28 ft/h; if $A_o > AW$, R = 1.0 ft/h for that part of A_o that is greater than AW, and 0.28 ft/h for that part of A_o that is less than AW.

Table 5.—(Continued)

TIME[1]	SX[11]	2SX OR (SX+W)[12]	A2SX OR ASX+AW[13]	AX[14]	AXRT[15]	AVG[16]	TOT (RUN+W)[17]	TOT RUN[18]
0	91047	166047	44266	22516	571	286	165762	90762
5								
0	68606	137212	42783	21391	499	250	143357	68357
5	136464	211464	46312	24562	742	620	211342	136342
10								
0	57065	114130	41488	20744	484	242	131823	56823
5	113404	188404	45333	23583	660	572	188316	113316
10	169721	244721	47871	26121	872	766	244615	169615
15								
0	39753	79506	39589	19795	462	231	114522	39522
5	78814	153814	43682	21932	523	492	153783	78783
10	118014	193014	45553	23803	679	601	192936	117936
15	235160	310160	51408	29658	3499	2767	309428	234428
30								
0	31931	63861	37800	18900	441	221	106710	31710
5	63200	126401	42176	21088	492	466	138175	63175
10	94613	169613	44436	22686	586	539	169567	94567
15	188603	263603	48925	27175	2879	2317	263041	188041
30	280962	355962	53605	31855	4049	3460	355373	280373
45								
0	26288	52577	36019	18009	420	210	101078	26078
5	51947	103894	40935	20467	478	448	126918	51918
10	77729	152729	43630	21880	518	498	152708	77708
15	155019	230019	47068	21990	2415	1985	229589	154589
30	231043	306043	51202	29452	3448	2929	305524	230524
45	305948	380948	54709	32959	4325	3883	380506	305506
60								
0	17312	34624	31649	15825	369	185	92127	17127
5	34071	68142	38317	19158	447	408	109031	34031
10	50896	101793	40826	20413	476	462	125882	50882
15	101388	176388	44759	23009	1837	1633	176184	101184
30	151285	226285	46916	25116	2377	2106	226014	151014
45	200575	275575	49594	27884	3046	2710	275239	200239
60	395816	470816	58315	36565	20905	16535	466446	391446
120								

11/ Preliminary estimate of volume of runoff stored in basin at end of time interval t_i, in cubic feet (S_x in equation 2).

12/ If $S_x \leq W$, S_o is doubled; if $S_x > W$, W is added to S_o.

13/ If $S_x \leq W$, infiltration area for doubled S_o is determined from volume/area curve (fig. 4), and is then halved to obtain infiltration area for S_x; if $S_x > W$, infiltration area for $S_o + W$ is determined from volume/area curve (fig. 4), and the infiltration area for W is subtracted to obtain the infiltration area for S_x.

14/ The net infiltration area for storm runoff, in square feet (A_x in equation 3).

15/ Second estimate of volume of runoff infiltrated during t_i, in cubic feet. If $A_x \leq$ AW, infiltration rate R = 0.28 ft/h; if $A_x >$ AW, R = 1.0 ft/h for that part of A_x that is greater than AW, and 0.28 ft/h for that part of A_x that is less than AW.

16/ Average volume of water infiltrated during t_i, in cubic feet (I_i in equation 1 is equivalent to $\frac{A_o \cdot R \cdot t_i + A_x \cdot R \cdot t_i}{2}$).

17/ Total volume of water in basin (runoff + reclaimed water) at end of time interval t_i, in cubic feet.

18/ Total volume of storm runoff in basin at end of time interval t_i, in cubic feet.

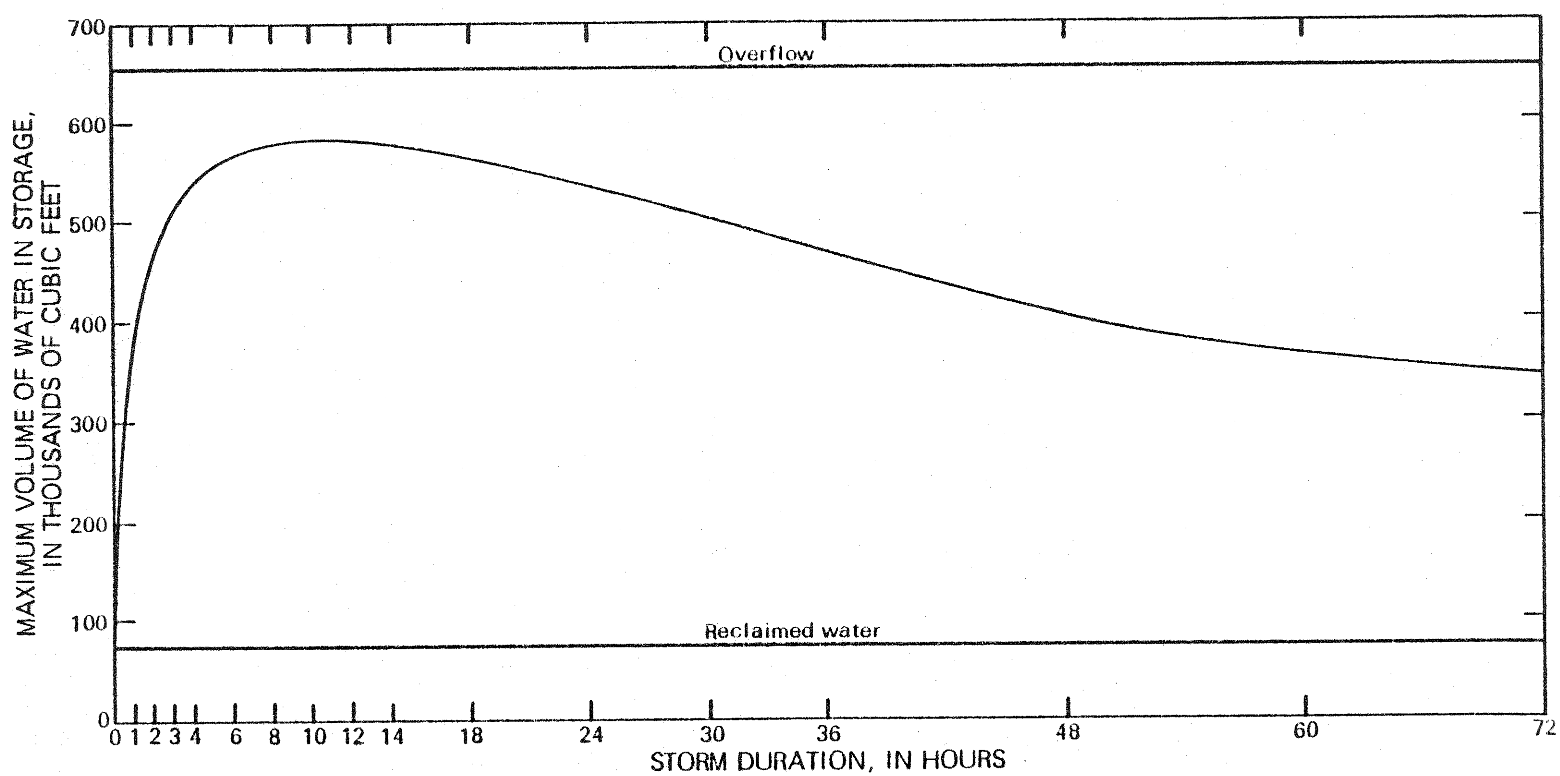

Figure 6.--Maximum volume of water in storage in basin 41 for any 10-year storm of 0- to 72-hour duration, infiltration rate of 0.28 ft/h below and 1.0 ft/h above 5-ft partition, and runoff coefficient of 40 percent.

Figure 6 also shows that, at an infiltration rate of 0.28 ft/h below the top of the 5-ft partition, basin 41 can accommodate substantial amounts of storm runoff, in addition to reclaimed water, without overflowing. However, a reduction in infiltration rate below the top of the partition would lower the reserve capacity of the basin. This effect can be shown by plotting maximum basin storage as a function of storm duration for a range of infiltration rates. Graphs of maximum basin stage during 10-year storms of different durations, at infiltration rates ranging from 0 to 1.12 ft/h below and 1.0 ft/h above the top of the partition, are shown in figure 7. The basin is shown to have sufficient storage capacity to store all runoff from 10-year storms of 0- to 72-hour durations regardless of the infiltration rate below the top of the partition, although at very low or zero infiltration rates, runoff storage closely approaches basin capacity.

As indicated in figure 7, the storm duration needed to produce maximum water storage in the basin increases as infiltration rate decreases. At an infiltration rate of 0.28 ft/h, the 10-year storm that produces maximum stress (peak storage) has a 10-hour duration. At the following reduced infiltration rates below the top of the partition, 10-year storms of the respective durations shown below produce maximum volumes of stored water.

Infiltration rate (feet/hour)	Worst-storm duration (hours)
0.14	13
0.07	16
0.03	17
0	18

If several graphs of the type shown in figure 7 are plotted for various combinations of infiltration rates and storm-return periods, the maximum values of water storage from each of the graphs can be used to develop graphs that correlate maximum water storage, infiltration rate, and storm-return period. Figure 8 is such a graph for basin 41; it shows the relationship between infiltration rate below the top of the partition and resulting maximum water storage during the most severe 10-, 25-, 50-, and 100-year storms at a runoff coefficient of 40 percent. As the figure indicates, basin 41 will not overflow during a 10-year storm regardless of infiltration rate. However, during 25-, 50-, and 100-year storms, infiltration rates below the top of the partition could not decrease to less than 0.56 ft/h, 1.38 ft/h, and 2.05 ft/h, respectively, without causing overflow.

The preceding analyses used a runoff coefficient of 40 percent. To evaluate the importance of runoff coefficient in the suitability of basins for artificial recharge, graphs were prepared for each of the 14 test basins to show the relationship between runoff coefficient and volume of water storage provided during large-magnitude storms. Infiltration rates below and above the top of the partition were 0.28 ft/h and 1.0 ft/h, respectively. Figure 9, which shows the relationship between runoff coefficient and maximum water storage at basin 41 during 10-, 25-, 50-, and 100-year storms, indicates that maximum allowable runoff coefficients during 10-, 25-, 50-, and 100-year storms are 46 percent, 37 percent, and 27 percent, respectively.

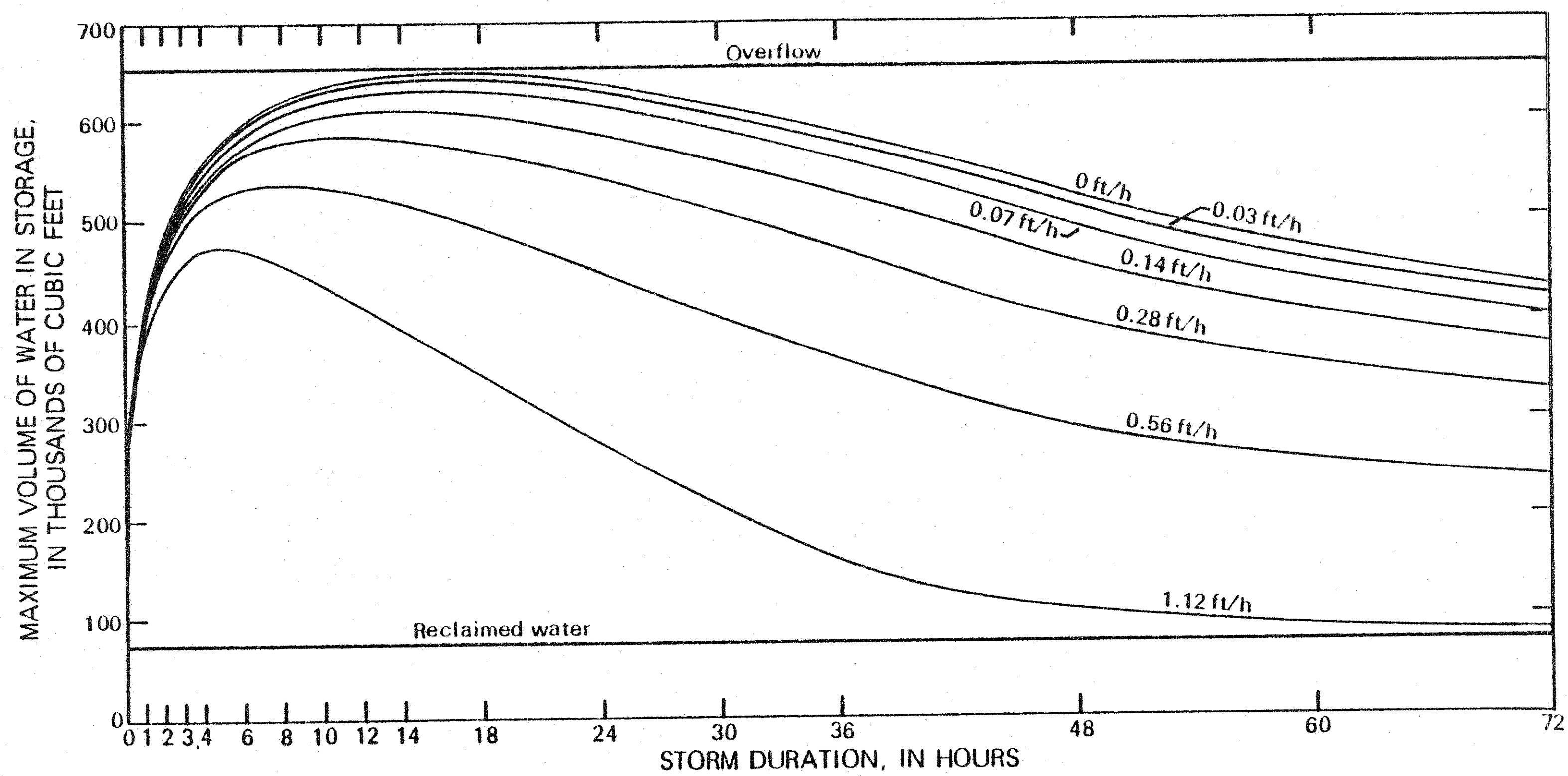

Figure 7.--Maximum volume of water in storage in basin 41 for any 10-year storm of 0- to 72-hour duration, infiltration rate of 0 to 1.12 ft/h below and 1.0 ft/h above 5-ft partition, and runoff coefficient of 40 percent.

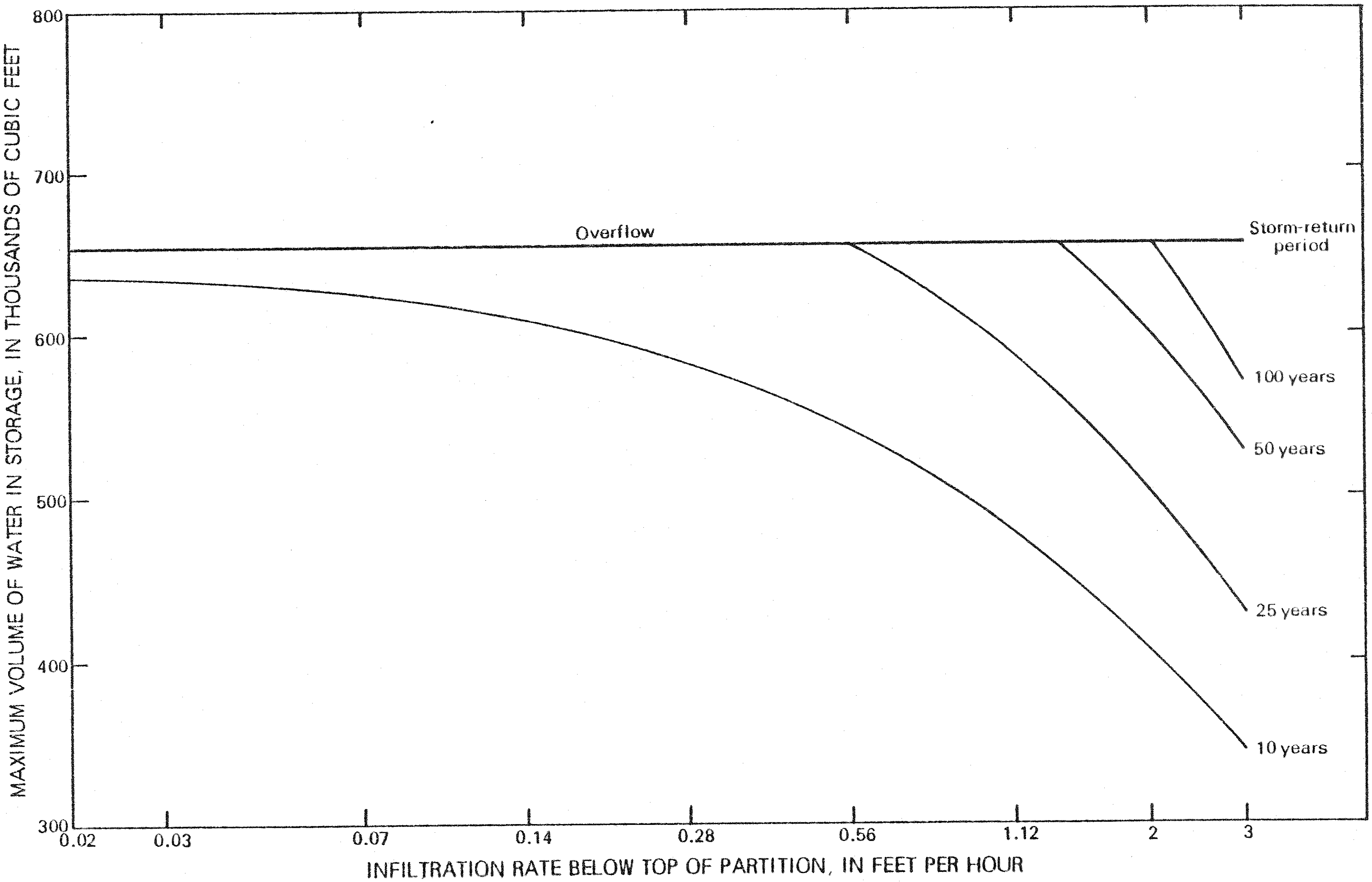

Figure 8.--*Maximum volume of water in storage basin 41 with different infiltration rates and storm-return periods, with reclaimed water ponded in half the basin to a depth of 5 ft, and runoff coefficient of 40 percent.*

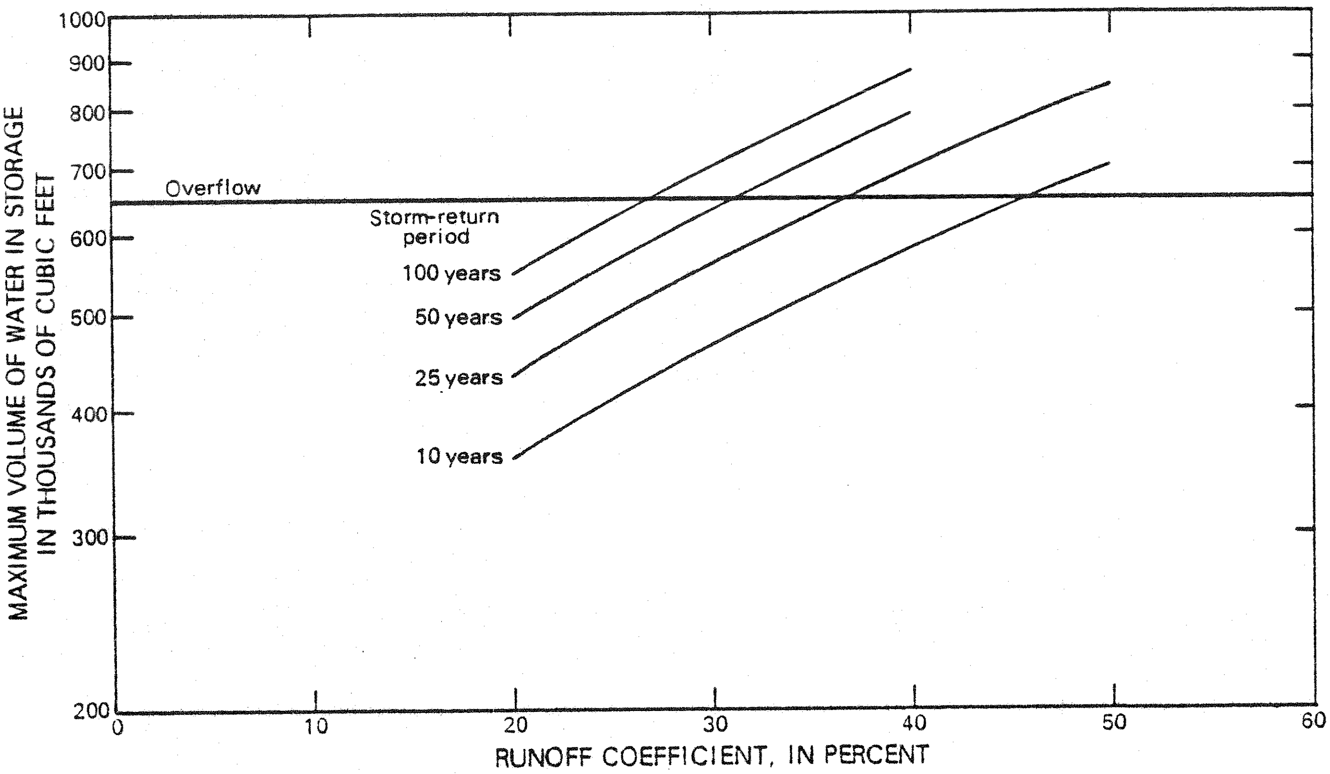

Figure 9.--Maximum volume of water in storage in basin 41 with different runoff coefficients and storm-runoff periods, with infiltration rates of 0.28 ft/h below and 1.0 ft/h above the partition and reclaimed water ponded in half the basin to a depth of 5 ft.

If runoff coefficients in effect during a storm are less than these, basin 41 will not overflow at an infiltration rate of 0.28 ft/h below the partition, regardless of storm duration or storm return period.

The preceding discussion demonstrates how equations 1 through 3 can be used to evaluate the suitability of a given basin for artificial recharge. The data in table 5 can supply information needed to evaluate other factors relating to the 14 selected basins. For example, if A_o and A_x (table 5) are averaged for each time interval of a storm, the average infiltration area A_i available for storm runoff during each time interval can be determined.

By weighting each value of A_i according to the duration of the time interval used to determine A_i, the average of the sum-weighted values of A_i will yield the average infiltration area available for storm runoff during the entire storm duration, as defined by the equation

$$\bar{A} = \frac{\sum_{i=1}^{n} \left(A_i \cdot \frac{t_i}{5}\right)}{n} \qquad (4)$$

where:

$\bar{A}$ is the average infiltration area available for runoff during the storm;

$t_i/5$ is the relative length of time interval used to determine A_i (t_i is measured in multiples of 5 minutes);

n is the number of A_i values selected.

Thus, if values of A_o, A_x, and t_i during the 2-hour storm listed in table 5 are used, a table such as the following can be prepared and used to calculate average infiltration area during the storm:

Infiltration Area (ft^2)			Time Interval (min)		Weighted A_i value (ft^2)
1/ A_o (eq. 2)	1/ A_x (eq. 3)	2/ A_i (eq. 3)	t_i	$\frac{t_i}{5}$ (eq. 4)	($A_i \cdot t_i$)
0	15,825	7,911	0-5	1	7,912
15,772	19,158	17,465	5-10	1	17,465
19,154	20,413	19,738	10-15	1	19,738
20,412	23,009	21,710	15-30	3	65,132
23,000	25,166	24,083	30-45	3	72,249
25,155	27,844	26,500	45-60	3	79,498
27,825	35,565	32,195	60-120	12	386,340
TOTALS --	--	--	--	24	648,334

1/ From 2-hour storm analysis in table 5.
2/ Average infiltration area during time interval, equal to $\frac{A_o + A_x}{2}$ (eq. 3)

From the total weighted A_i values divided by the total 5-min time intervals, $\bar{A}$ is computed as follows:

$$\bar{A} = \frac{648{,}334}{24} ft^2 = 27{,}014 ft^2$$

The average area infiltrated by storm runoff during the sample storm is 27,014 ft^2. If this infiltration area is added to the 21,800-ft^2 area infiltrated by reclaimed water (table 4), the total infiltration area for combined runoff and reclaimed water is 48,814 ft^2. From figure 4, this area corresponds to a water depth of 7.3 ft, which indicates that some runoff infiltrated the basin walls above the 5-ft partition. Average infiltration areas during the storms of greatest stress at each of the 14 test basins were determined in this manner and are given in table 6.

Table 6.--Summary of basin-performance analyses for 14 test basins

Basin	Storm-return period (years)	Storm duration of maximum stress[1] (h)	Cumulative runoff to basin (ft^3)	Maximum volume of stored water: Runoff and reclaimed water[2] (ft^3)	Maximum volume of stored water: Runoff[2] (ft^3)	Volume of runoff infiltrated during storm (ft^3)	Percentage of storage capacity used[3]
41	10	10	660,200	581,600	506,600	153,600	89
	25	8	760,800	695,600	620,600	140,200	107
	50	10	909,400	793,100	718,100	191,300	121
	100	10	1,001,800	876,900	801,900	199,900	134
51 East	10	6	1,489,200	1,403,000	1,110,000	379,200	67
	25	6	1,825,100	1,709,400	1,416,400	408,700	81
	50	8	2,239,700	1,946,200	1,653,200	586,500	93
	100	8	2,468,400	2,153,000	1,860,000	608,000	103
51 West	10	6	1,303,800	1,378,400	953,400	350,400	79
	25	6	1,608,400	1,602,200	1,177,200	431,200	92
	50	4	1,668,900	1,779,300	1,354,300	314,600	102
	100	4	1,849,600	1,934,300	1,509,300	340,300	111
62	10	10	2,615,600	2,307,800	1,922,800	692,800	36
	25	8	3,014,400	2,790,400	2,405,400	609,000	44
	50	8	3,438,700	3,168,900	2,783,900	654,800	50
	100	10	3,969,100	3,486,700	3,101,700	867,400	55
223	10	10	587,700	565,900	433,900	153,800	83
	25	8	677,300	670,000	538,000	139,300	100
	50	8	772,700	753,800	621,800	150,900	111
	100	8	851,600	825,000	693,000	158,600	125
244	10	8	615,700	574,100	468,100	147,600	78
	25	8	751,900	697,800	591,800	160,100	94
	50	8	857,800	792,200	686,200	171,600	107
	100	8	945,400	879,400	773,400	172,000	119
260	10	10	601,200	638,800	436,800	164,400	91
	25	8	692,900	750,900	548,900	144,000	108
	50	8	790,400	837,900	635,900	154,500	120
	100	8	871,200	913,200	711,200	160,000	130
296	10	10	731,500	716,200	548,700	182,800	51
	25	10	876,400	843,800	676,300	200,100	61
	50	10	1,007,700	960,400	792,900	214,800	69
	100	10	1,110,000	1,051,500	884,000	226,000	76
312	10	10	505,600	484,700	377,700	127,900	80
	25	8	582,600	576,000	469,000	113,600	91
	50	10	696,500	649,200	542,200	154,300	103
	100	12	801,800	711,300	604,300	197,500	113
334	10	8	1,015,900	1,127,800	737,800	278,100	42
	25	8	1,240,800	1,324,700	934,700	306,100	50
	50	8	1,415,400	1,481,100	1,091,100	324,300	56
	100	8	1,560,000	1,610,200	1,220,200	339,800	60
358	10	10	1,322,700	1,259,400	947,400	375,300	65
	25	8	1,524,400	1,509,000	1,197,000	327,400	78
	50	8	1,739,000	1,705,200	1,393,200	345,800	88
	100	8	1,916,600	1,866,300	1,554,300	362,300	96
376	10	4	540,600	628,000	399,000	141,600	51
	25	8	663,300	735,400	506,400	153,900	60
	50	8	753,200	817,700	588,700	164,500	67
	100	8	830,200	886,500	657,500	172,700	72
413	10	14	2,690,600	2,461,500	2,090,500	600,100	85
	25	14	3,182,000	2,877,500	2,506,500	675,500	100
	50	14	3,673,300	3,303,500	2,932,500	740,800	114
	100	14	4,060,000	3,652,600	3,281,600	778,400	126
419	10	10	1,009,100	1,031,000	781,000	228,100	81
	25	12	1,256,100	1,204,800	954,800	301,300	94
	50	12	1,445,200	1,371,900	1,121,900	323,300	107
	100	12	1,600,500	1,509,000	1,259,000	341,500	118

1/ Storm durations were rounded to values nearest those listed in table 3.
2/ Includes volume of overflow.
3/ Values greater than 100 indicate overflow.

Table 6.--Summary of basin-performance analyses for 14 test basins (Continued)

Basin	Volume of overflow during storm (ft³)	Ratio of volume of overflow to total volume of runoff (percent)	Average infiltration area used for runoff (ft³)	Effective average infiltration rate for runoff (ft/h)	Total duration of storm-runoff storage 4/ (h)	Minimum infiltration rate required to prevent overflow 5/ (ft/h)	Maximum allowable runoff coefficient required to prevent overflow 6/
41	0	0	30,650	0.50	54	0	46
	42,600	6	32,840	.53	56	.56	37
	140,100	15	34,540	.55	58	1.38	31
	233,900	22	35,580	.56	58	2.05	27
51 East	0	0	125,220	.50	36	0	58
	0	0	130,670	.52	40	0	49
	0	0	136,380	.54	45	.04	43
	53,000	2	139,140	.55	47	.37	39
51 West	0	0	149,470	.39	29	0	50
	0	0	165,540	.43	31	.05	44
	40,900	2	172,820	.46	30	.40	39
	195,900	11	180,650	.47	30	.73	36
62	0	0	167,620	.41	44	0	94
	0	0	175,220	.43	46	0	81
	0	0	181,600	.45	49	0	73
	0	0	186,940	.46	55	0	66
223	0	0	41,330	.37	40	0	47
	0	0	43,590	.40	42	.24	40
	73,800	10	45,140	.42	42	.57	34
	145,000	17	46,170	.43	42	1.03	30
244	0	0	52,660	.35	37	0	53
	0	0	54,740	.37	40	.18	43
	52,200	6	56,000	.38	41	.47	37
	139,400	15	56,560	.38	41	.70	32
260	0	0	52,650	.31	36	.15	44
	50,900	7	54,320	.33	35	.43	37
	137,900	17	55,690	.35	35	.78	32
	213,200	24	56,560	.35	35	1.20	29
296	0	0	56,250	.32	39	0	70
	0	0	58,520	.34	41	0	61
	0	0	60,260	.36	44	0	54
	0	0	61,560	.37	47	0	50
312	0	0	36,830	.35	40	0	55
	0	0	38,580	.37	42	.12	44
	19,200	3	39,640	.39	46	.35	39
	81,300	10	40,630	.41	48	.64	35
334	0	0	111,660	.31	30	0	93
	0	0	117,690	.32	33	0	80
	0	0	120,900	.34	34	0	72
	0	0	123,900	.34	36	0	66
358	0	0	117,180	.32	36	0	63
	0	0	121,620	.34	37	0	53
	0	0	124,120	.35	39	.12	46
	0	0	127,060	.36	41	.23	42
376	0	0	60,450	.29	26	0	80
	0	0	62,710	.31	33	0	69
	0	0	64,580	.32	35	0	62
	0	0	66,110	.33	36	0	57
413	0	0	93,570	.46	60	0	50
	0	0	98,710	.49	66	.27	40
	412,500	11	103,110	.51	66	.68	33
	761,600	19	105,580	.53	66	.98	28
419	0	0	60,450	.29	40	0	51
	0	0	62,700	.31	46	.13	43
	91,900	6	64,580	.32	48	.43	38
	229,000	14	66,110	.33	48	.60	33

4/ Time from start of precipitation to end of runoff storage in basin, in hours.

5/ Values refer to minimum required infiltration rate below 5-ft partition with a runoff coefficient of 40 percent. A value of 0 indicates that overflow would not occur if infiltration rate below 5-ft partition declined to zero.

6/ Values refer to maximum allowable runoff coefficient to prevent overflow of runoff, at infiltration rates of 0.28 ft/h below and 1.0 ft/h above 5-ft partition.

With knowledge of average infiltration area at a given basin during a storm, the average infiltration rate $\overline{R}$ (in ft/h) in effect during the storm can be calculated by the equation

$$\overline{R} = \frac{\sum_{i=1}^{n} I_i}{\overline{A} \cdot d} \qquad (5)$$

where d is duration of storm, in hours.

For the 120-minute storm at basin 41 (table 5), total volume of infiltration represented by the sum of average volumes of infiltration during each time interval equals 24,038 ft^3 1/, and, from equation 5, average infiltration rate at basin 41 is 0.44 ft/h. Average infiltration rates during the storms of greatest stress at each of the test basins are given in table 6.

Total duration of storm-water storage in a basin during and after a storm was determined from equation 1. At the end of the storm, the volume of incremental runoff to the basin (V_i in eq.1) was assigned a value of zero. Incremental declines in volume of stored runoff were calculated at 60-minute increments until the volume of stored runoff declined to zero. The time required for all storm runoff to infiltrate after inflow had ceased was then added to the storm duration to obtain total duration of storm-runoff retention. Retention times, rounded to the nearest hour, are listed in table 6.

Table 6 summarizes the results of analyses of the operating characteristics of the 14 test basins during idealized high-intensity storms having 10-, 25-, 50-, and 100-year return periods. In the following sections, the significant results are discussed in terms of the column headings in table 6.

Storm duration of maximum stress--As shown in table 6, the storm duration for a given storm-return period that produces maximum stress on a basin differs from basin to basin. For example, durations of as little as 4 hours during a 50- or 100-year storm will produce the maximum stress on basin 413 regardless of return period. Differences between duration of the most stressing storm of a given return period at each basin reflect the differences in volume of runoff to each basin and differences in the volume/area relationships of each basin.

1/ $\sum_{i=1}^{n} I_i$ is equivalent to the net difference between cumulative volume of runoff throughout the storm and the final volume of stored runoff. From the 2-hour storm in table 5,

$$\sum_{i=1}^{n} I_i$$

equals 415,484 ft^3 minus 391,446 ft^3; therefore, 24,038 ft^3 of runoff infiltrated during that storm.

Cumulative runoff to basin--Cumulative runoff to a basin is the total volume of water falling on the basin's drainage area, resulting from precipitation of a duration and intensity specified for that particular storm duration and return period (listed in table 3).

Maximum volume of stored water--Maximum volume of stored water in a basin is the sum of storm inflow (cumulative runoff) and reclaimed water entering the basin, less the amount of storm runoff that infiltrates the basin during the storm. This category includes the volume of water that would overflow the basin if total storage should exceed the basin's storage capacity.

Volume of runoff infiltrated during storm--These values represent the difference between volume of cumulative runoff to the basin and the maximum volume of runoff stored in the basin during a storm. Maximum volume of stored storm runoff, in turn, is the difference between maximum volume of stored water and volume of reclaimed water in the basin, and includes volume of overflow.

Percentage of storage capacity used--These values are the ratio of the maximum volume of stored water (runoff and reclaimed water) to the total storage capacity of the basin and is an indication of the reserve capacity of the basin during supplemental recharge. The basin with the largest reserve capacity for impounding storm runoff and reclaimed water is basin 62, where only 36 to 55 percent of the storage capacity is used during 10- to 100-year storms. (See table 2). The basin with the smallest reserve capacity is basin 41, where 89 to 134 percent of total storage capacity is used by stored water during 10- to 100-year storms.

Volume of overflow during storm--The volume of overflow from a basin during a storm of specified duration and return period represents the sum of maximum volume of stored runoff and reclaimed water less the storage capacity of the basin. As shown in table 6, only basins 62, 296, 334, 358, and 376 would not overflow during large-magnitude storms with simultaneous infiltration of reclaimed water; the nine remaining basins would overflow during 25-, 50-, or 100-year storms (table 6). None of the 14 test basins would overflow during a 10-year storm.

Ratio of volume of overflow to total volume of runoff--The volume of water that overflows a basin during a large-magnitude storm depends on infiltration characteristics of the basin, volume of storm runoff entering the basin, and the basin's volume/area relationship. To facilitate comparison of performance characteristics among the selected basins and to minimize effects of differences in drainage area, storm magnitude, and resulting amount of runoff, the ratio between volume of overflow and total volume of runoff entering the basin was evaluated. Of the basins in which overflow would occur, basin 41 would produce the largest ratio of overflow to volume of total runoff (table 6), which suggests that this basin is the least efficient of the 14 selected for supplemental recharge during large-magnitude storms.

The preceding analysis can be used to assess the possibility of diverting overflow to other, nearby storm-water basins. Analyses by Aronson and Prill (1977) and Aronson (1976b) have shown that typical storm-water basins on Long Island could accept much larger volumes of storm runoff than the design

capacity of the basins would indicate and that overflow from certain basins could readily be accommodated by other nearby basins used solely for disposal of storm runoff. Moreover, volumes of overflow listed in table 6 could be effectively eliminated if the basins in question were enlarged by an amount equivalent to the volume of overflow. As an example, if basin 51 West were excavated an additional 2.1 ft in depth, approximately 196,000 ft^3 of additional storage capacity would be available, and, as a result, the basin would probably not overflow during a 100-year storm of any duration. Of course, if a basin is deepened, the volume/area relationship will change and thereby affect the final volume of stored water in the basin after a storm. Also, the volume of reclaimed water that could be stored within the partitioned area would be reduced somewhat by the inward slope of the basin sides.

Average infiltration area used for runoff--The average infiltration area (A_i) during each of the incremental time intervals (t_i) used to solve equations 1 through 3 was weighted according to the duration of t_i. (See section "Method of Basin Performance Analysis.") The mean value of the weighted average infiltration areas during the storm represents the average infiltration area for stored runoff during the storm. Average infiltration areas listed in table 6 were used to determine average infiltration rates of storm runoff and durations of runoff storage.

Effective average infiltration rate for runoff--The average infiltration rate of basin during a specified storm is determined by dividing (a) the amount of runoff lost through infiltration by (b) the average infiltration area during the storm. The maximum average infiltration rate (0.50 to 0.56 ft/h), depending on storm-return period, occurs at basin 41, whereas minimum average infiltration rates (0.29 to 0.33 ft/h) occur at basins 376 and 419. Inasmuch as the calculations were based on only two infiltration rates in all 14 basins--one above and one below the top of the 5-ft partition--any differences in average infiltration rates among basins, or among storms of differing intensity at the same basin, may be attributed to longer periods of water storage above the top of the partition, which, in turn, is related to the characteristic volume/area relationship of each basin.

Total duration of storm-runoff storage--This value is the time interval from the start of inflow to complete infiltration of impounded runoff, and represents the sum of storm duration and time required for the maximum volume of overflow) to infiltrate the basin. Calculated duration of storage is shortest at basin 51 West, where it ranges from 29 to 30 hours (depending on storm-return period), and is greatest at basin 413, where it ranges from 60 to 66 hours.

Calculated durations of storm-runoff retention indicate that runoff from the most severe 100-year storms will completely infiltrate even the least efficient basin within approximately 3 days after cessation of the storm, provided infiltration rates above and below the top of the partition remain constant. If infiltration rates should decline during impoundment of runoff, retention will be longer than indicated in table 6. Should another large-magnitude storm occur before all stored runoff has infiltrated the basin, the basin would probably overflow unless excess storm runoff or reclaimed water were diverted to another storage site.

As shown in table 6, durations of water retention at a given basin are the same during storms of similar length in which overflow occurs, even though total volumes of runoff to the basin differ. The reason is that a basin can hold only a specific amount of water, and overflow water will not infiltrate the basin. As a result, the time required for a specified volume of stored runoff to infiltrate a basin after a storm in which overflow occurs will be the same regardless of the volume of overflow.

Minimum infiltration rate required to prevent overflow--In the analyses of the operating characteristics of the test basins, infiltration rate and runoff coefficient were varied to determine their effects on the basins' ability to accommodate runoff and reclaimed water without overflowing. The runoff coefficient in these analyses was held constant at 40 percent, and infiltration rate below the top of the partition was varied from 0 to 3.0 ft/h. Graphs similar to that in figure 9 were prepared for each basin. Table 6 lists the minimum infiltration rates, below the top of the partition, that would prevent overflow at each of the test basins. As this column of table 6 indicates, the infiltration rate below the top of the partition of all but one of the selected basins (basin 260) could decline to 0 ft/h during a 10-year storm without overflow. Basins 62, 296, 334, and 376 would not overflow even during 100-year storms if infiltration rates below the top of the partition declined to zero. The majority of basins, however, require an infiltration rate greater than zero to prevent overflow during large-magnitude storms, and some basins, such as basins 41, 223, and 260, require infiltration rates of greater than 1.0 ft/h below the top of the partition to prevent overflow during 100-year storms. Although this rate is not high in relation to those observed at these basins, it might be difficult to achieve or maintain during prolonged ponding of runoff. However, the ability of the basin to accommodate the maximum runoff without overflowing could be increased by (1) enlarging the basin, as discussed under the heading "Ratio of volume of overflow to total volume of runoff"; (2) diverting excess runoff to another basin (see section "Description of Storm-Water Basins in Nassau County"); or (3) temporarily halting inflow of reclaimed water by diverting it to tide-water at the water-treatment plant, so that additional infiltration area would become available for storm runoff.

Maximum allowable runoff coefficient required to prevent overflow--Assuming respective infiltration rates of 0.28 ft/h below and 1.0 ft/h above the 5-ft partition, the runoff coefficient was varied for each test basin during analyses of basin operation during large-magnitude storms to determine the maximum allowable runoff coefficient that would still prevent overflow. Graphs similar to those in figure 8 were prepared for each basin; table 6 summarizes the results of these analyses. As shown in table 6, runoff coefficients at basins during large-magnitude storms could be much higher than the 40-percent design coefficient without causing overflow. For example, the runoff coefficient at basin 62 could be as high as 94 percent during a 10-year storm or 66 percent during a 100-year storm without producing overflow. In contrast, maximum allowable runoff coefficients at basin 41 range from 46 to 28 percent during 10- and 100-year storms, respectively. To determine whether the maximum allowable runoff coefficient calculated for each of the test basins would actually be exceeded during large-magnitude storms, runoff coefficients in effect at six of the test basins during the storms of August 8-9, 1976

(Hurricane Belle) were calculated by the method described in Aronson (1977). From infiltration rates measured during several storms at each of these six basins, runoff coefficients were calculated to be about 25 percent at basin 41, 30 percent at basin 51 West, 25 percent at basin 62, 30 percent at basin 296, 20 percent at basin 312, and 20 percent at basin 358. These values are all less than the maximum allowable coefficients listed in table 6 and indicate that these basins, as well as the other eight, probably would not overflow during most storms on Long Island.

Although runoff coefficients cannot be controlled during a storm, knowledge of their limiting values and the knowledge that they will rarely exceed the design coefficient during the large majority of storms on Long Island would support the use of the 14 selected basins for impoundment of reclaimed water and storm runoff.

Role of Partition Height

In the preceding analyses of the operating characteristics of the 14 selected test basins during artificial recharge, partition height within the basins was assumed to be 5 ft. With this partition height, several of the basins were shown to be prone to overflow during large-magnitude storms. The question thus arose as to whether such basins would overflow if partition height were lowered to increase storage volume and infiltration capacity for storm runoff by reducing the available storage capacity and infiltration area for reclaimed water. Analyses were performed to determine the effect of partition height on likelihood of overflow during large-magnitude storms. Analyses were performed at basin 41 with partition heights ranging from 0 to 6 ft; results of these analyses are shown in figure 10. The analyses were made using worst-storm rainfalls. Infiltration rates of 0.28 ft/h below and 1.0 ft/h above the partition were assumed with a runoff coefficient of 40 percent. As figure 10 indicates, basin 41 would not overflow during a 10-year storm even if partition height exceeded 6 feet, and the basin would not overflow during 25-, 50-, and 100-year storms as long as partition height did not exceed 3.2 ft, 1.5 ft, and 0.6 ft, respectively.

Analyses of the role of partition height or volume of stored water during large-magnitude storms were not made for the other 13 test basins; however, the relationship of partition height to a basin's ability to dispose of storm runoff and reclaimed water without overflowing would be expected to be similar in basins with performance characteristics similar to those of basin 41.

When reclaimed water is applied to half of a partitioned basin at a constant rate lower than the infiltration capacity of the basin, it will form only a thin layer over the spreading area. As infiltration rate decreases with reduced soil permeability after continued water application, however, the water level will begin to rise. The rate of rise will be partly offset by an increase in infiltration rate that accompanies the increase in hydraulic head at the soil/water interface as water depth increases, but the water level will still rise at a rate sufficient to increase the infiltration area as the infiltration rate declines; thus, the infiltration capacity will

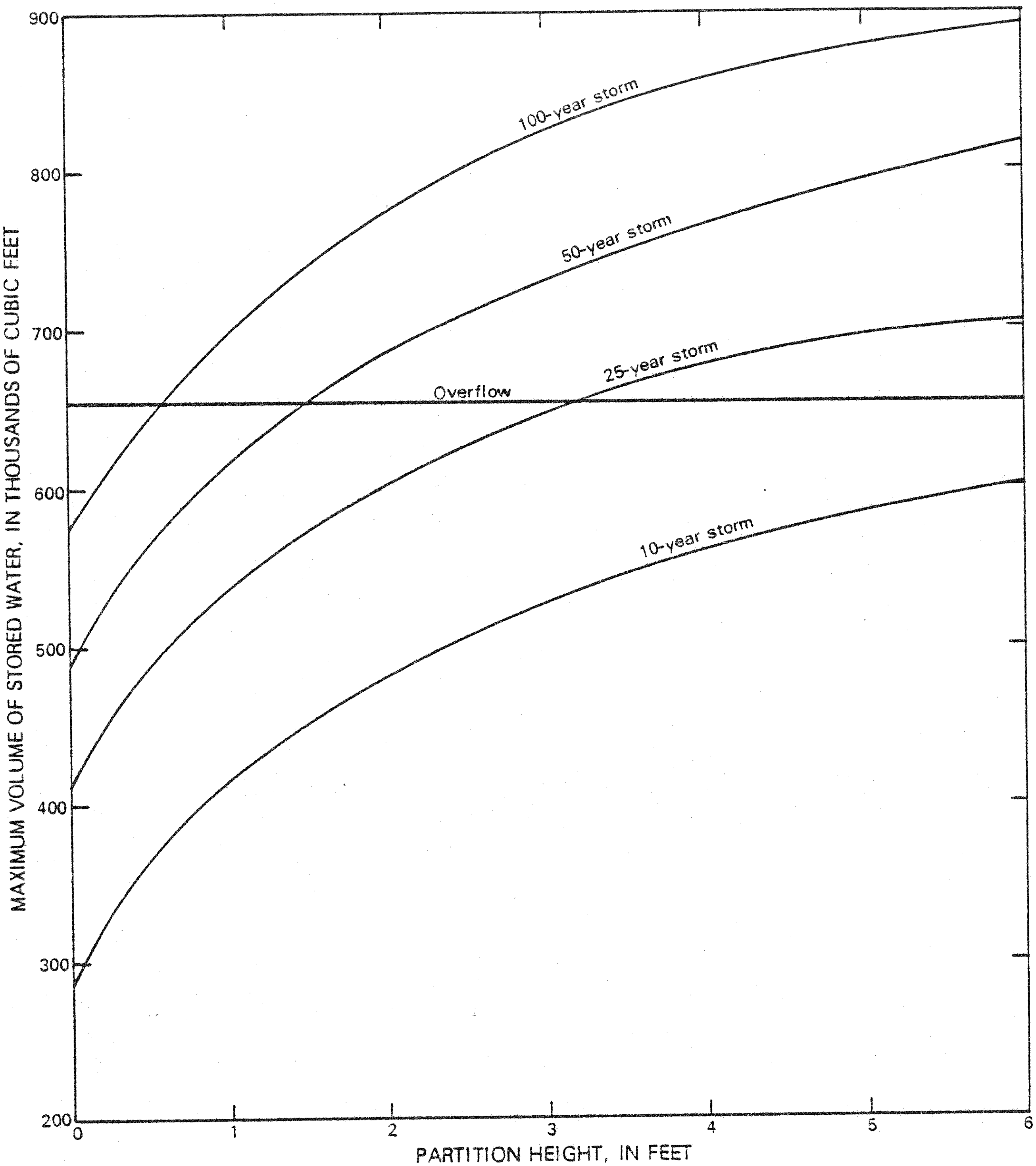

Figure 10.--Maximum volume of water in storage in basin 41 at various partition heights and storm-return periods. Infiltration rate is 0.28 ft/h below and 1.0 ft/h above top of partition; runoff coefficient is 40 percent.

remain constant as the basin fills. When the water level reaches the top of the partition, the maximum infiltration capacity of half of the basin has been reached and application of reclaimed water must then be switched to the other basin half (fig. 11). Thus, at a given ratio of reclaimed water inflow to available infiltration area, partition height governs the duration of the application and rest periods.

It was shown that basin 41 could accommodate runoff from a large-magnitude storm, regardless of storm duration or return period, if partition height were lowered to make additional infiltration area available for storm runoff. Basins that would overflow during large-magnitude storms (table 6) might not do so if partition height, and hence duration of application and rest periods, were reduced. Of course, lowering the partition reduces the infiltration area available for reclaimed water, which, in turn, decreases the volume of reclaimed water that can be applied during a given period.

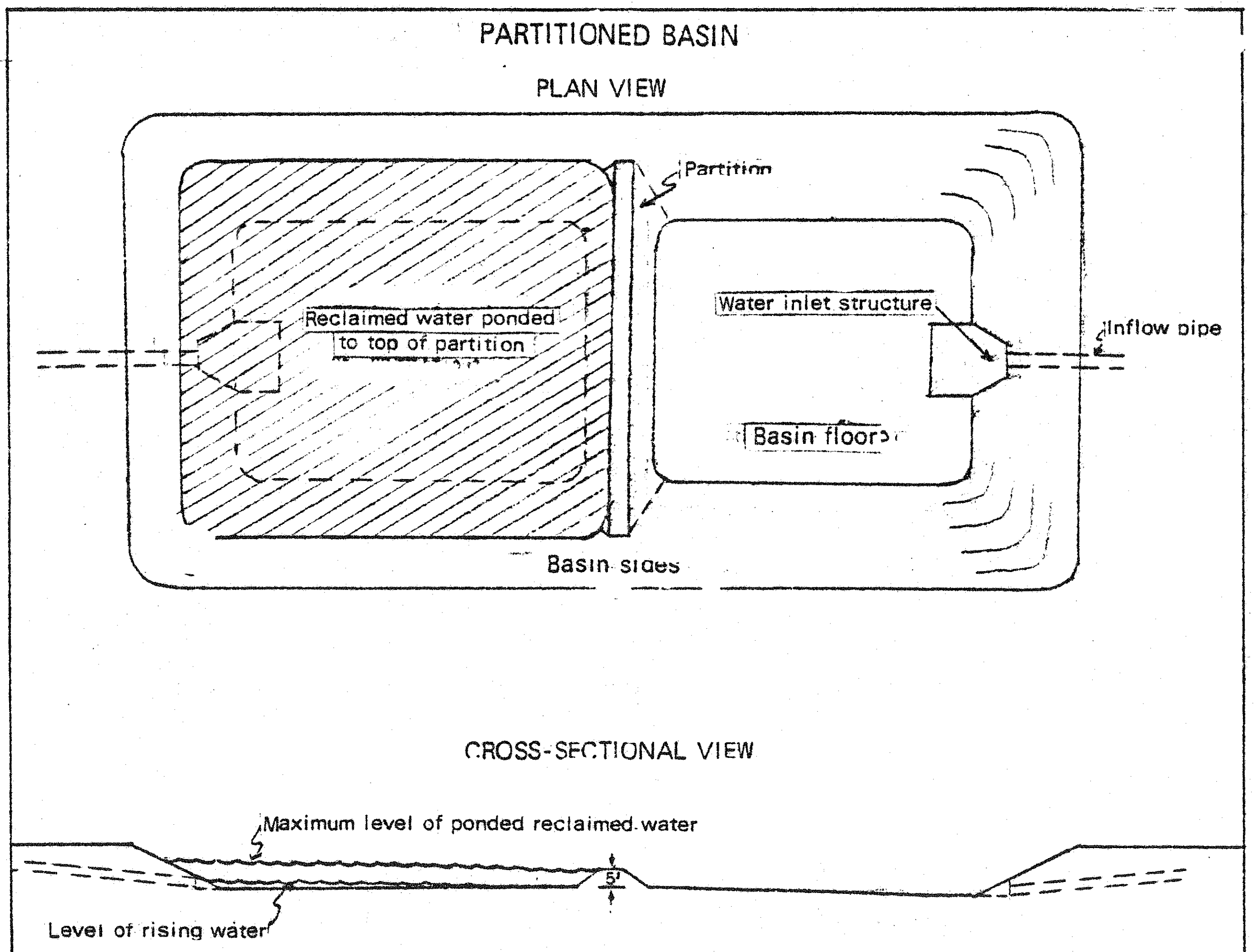

Figure 11.--Plan view and cross section of a storm-water basin showing rising and maximum levels of ponded reclaimed water.

Summary of Results

Results of the preceding analyses indicate that all 14 test basins studied can be partitioned to accommodate reclaimed water and runoff from large-magnitude storms if certain conditions are met. Basins 62, 296, 334, 358, and 376 (fig. 2) would not overflow during 10-, 25-, 50-, or 100-year storms, regardless of storm duration, at infiltration rates of 0.28 ft/h below and 1.0 ft/h above the top of the partition. Of these five basins, only basin 376 would overflow during a 100-year storm, even if infiltration rate below the 5-ft partition declined to zero. At basins that would overflow during certain large-magnitude storms, volume of overflow compared to volume of inflow is relatively small and could be piped to other nearby storm-water basins.

The analyses presented here indicate that progressive urbanization within the drainage area of most recharge basins and the attendent increase in volume of storm runoff will not significantly alter basin performance as long as periodic maintenance of the basins can maintain infiltration at rates at least as high as those used in the analyses. However, some of the basins would overflow during extremely large-magnitude storms if runoff were significantly greater than expected because of frozen soil or urbanization of the basins' drainage areas. Overflow of these basins could be prevented by (1) enlarging the basin; (2) restricting the inflow of reclaimed water; or (3) limiting the partition height.

Although infiltration rates of 0.28 ft/h below and 1.0 ft/h above a 5-ft partition and a runoff coefficient of 40 percent were assumed in the preceding analyses, published and unpublished studies of several storm-water basins and Nassau and Suffolk Counties have shown that infiltration rates of basin floors generally are considerably higher and average about 1.0 ft/h (Seaburn and Aronson, 1973; Prill and Aronson, 1978). If a high rate of infiltration could be maintained by scarifying or tilling the basin floor during the rest phase of the application-and-rest cycle, the likelihood of overflow during large-magnitude storms would be considerably reduced. Similarly, studies have shown that the runoff coefficient of 40 percent may be conservative and generally averages less than 30 percent on Long Island (Seaburn and Aronson, 1974; Aronson, 1977). Furthermore, the preceding basin-performance analyses were based on storm durations that produce the greatest stress on the basins. Under real conditions, worst-storm conditions occur only rarely; thus, the results of this study are conservative.

The duration of containment of storm runoff is the only real limitation of the use of storm-water basins for supplemental recharge with reclaimed water. Results of analyses listed in table 6 show that runoff remains in several of the test basins for almost 3 days after a storm. If another large-magnitude storm should occur before a basin has emptied, the basin might overflow. However, in such an eventuality, either runoff or reclaimed water could be temporarily diverted to prevent overflow.

The duration of infiltration and rest periods during reclaimed-water application will depend on (1) the infiltration capacity of the basin, (2) the rate at which infiltration rate declines with continued ponding, and (3) the height of the partition between basin halves. If partition

height is kept sufficiently low, overflow will be unlikely during even the most severe storms. Reduction of partition height will also shorten the duration of the application and rest cycle, however, and will also reduce the area available for infiltration of reclaimed water.

The 14 selected test basins have been shown capable of accepting both reclaimed water and runoff from large-magnitude storms with little likelihood of overflow. Enlarging certain basins, limiting partition height, or diverting excess storm runoff are some of the ways by which supplemental recharge could be sustained at these basins without danger of overflow.

WATER-TABLE RESPONSE AS A LIMITATION ON RATES OF RECHARGE

Introduction

Although the 14 selected storm-water basins can dispose of runoff from large-magnitude storms with little likelihood of overflow, their ability to dispose of runoff on a continuing basis is dependent on factors other than available infiltration area and infiltration rate. After infiltrating a basin floor, runoff percolates downward to the water table to form a mound, the height of which is dependent on (1) the amount and rate of water reaching the water table, and (2) hydrologic characteristics of the deposits underlying the basin.

At any given recharge site, rate of recharge should be such that the water-table mound will not rise high enough to intersect the basin floor. The maximum permissible recharge rate at a basin site can be evaluated with models of the ground-water reservoir, which can simulate the change in the water-table altitude locally and regionally when storm-water basins are used for artificial recharge. From knowledge of maximum permissible recharge rates and the characteristics of the water-table mound associated with those rates, a suitable recharge scheme can be divided to maximize the efficiency of the recharge operations.

A recharge scheme based on rises in the ground-water reservoir during artificial recharge was determined from two models--a digital simulation for local effects, and an analog simulation for the regional effect. Correlation of information from both models should provide accurate estimates of the water-table rise that would result from basin recharge and can be used to determine the maximum recharge rates that are feasible at the 14 selected basins without causing excessive water-table mounding and basin flooding.

Method of Analysis

Two models were used to simulate the recharge basins because a single model with sufficient resolution over the entire area of study would be prohibitively complex and expensive. One, termed the local model, has high resolution but restricted areal representation. It is accurate in determining the mound configuration beneath and near individual basins. In contrast, the regional model represents all of Long Island except the eastern fork, so that recharge through all 14 test basins may be simulated at once. Although the resolution of this model is too low to predict results near a single basin, the two models together can provide an accurate analysis of conditions within and around the study area.

Local Model

A local digital model was designed to simulate the development of a water-table mound beneath each of the 14 test basins during artificial recharge. To accurately model the basins, certain hydrologic and geologic characteristics of the underlying deposits had to be determined and incorporated. Lithologic samples and gamma-ray, spontaneous-potential, and resistivity logs of the 0- to 102-ft depth beneath each basin were analyzed to locate clay lenses or clay-bearing deposits of low hydraulic conductivity within the upper glacial aquifer and upper part of the Magothy aquifer that might retard downward movement of reclaimed water to the ground-water reservoir (Aronson, 1976a). The aquifer material under the basins is stratified and, for purposes of modeling, was assumed to be radially symetric about the center of each basin. Lithologic logs of wells near the test basins were compiled and analyzed to determine the geologic and hydrologic character of deposits deeper than 100 ft. Horizontal and vertical hydraulic conductivities of the deposits, also needed for the model analysis, were assigned to the several lithologic layers beneath each basin, as determined by McClymonds and Franke (1972) and Franke and Cohen (1972). Table 7 lists pertinent hydrogeologic characteristics of deposits beneath each of the 14 test basins.

Water-table mounding beneath the basins during artificial recharge was analyzed by a Galerkin finite-element model, designed to produce a "steady-state" solution in which effects of recharge on water-table mounding have stabilized over time. The model uses elements that represent triangular sections of the aquifer beneath each basin. The mathematical solution for ground-water flow in each section yields an approximate solution for the real system. The model simulates recharge, multiaquifer systems, and anisotropic conditions radially and vertically. The model is defined by steady-state ground-water flow equations. Background information for developing the model was obtained from Zienkiewicz (1971), Desai and Abel (1972), and Pinder and Gray (1977). A typical model grid is shown in figure 12. The model assumes a constant-head boundary at its radial extremities; its base is assumed to be a no-flow boundary at the top of the Raritan clay (base of the Magothy aquifer). (See table 1.) Because reclaimed water is ponded in only half of a given test basin, the local model predicts the configuration of a water-table mound beneath and radially outward from the center of the half basin in use (fig. 1). For ease of analysis, rectangular basin halves were considered to be circles of equivalent area.

Table 7.--Hydrogeologic data used to model local water-table mound beneath each test basin

Basin	Depth to water table (ft)1/	Aquifer	Depth interval below basin floor (ft)	Ratio of horizontal to vertical hydraulic conductivity [(gal/d)/ft²]
N41	30	unsaturated upper glacial	0-30	2/ 2500/250
		upper glacial	30-75	2500/250
		Magothy	75-650	500/13.9
N51 East and N51 West	20	unsaturated upper glacial	0-20	2/ 2500/250
		upper glacial	20-40	2500/250
		Magothy	40-620	700/19.4
N62	15	unsaturated upper glacial	0-15	2/ 2500/250
		upper glacial	15-30	2500/250
		Magothy	30-190	1000/100
			190-630	200/5.6
N223	45	unsaturated upper glacial	0-40	2/ 2500/250
		unsaturated Magothy	40-45	2/ 600/16.7
		Magothy	45-675	600/16.7
N244	90	unsaturated upper glacial	0-90	2500/250
		upper glacial	90-105	2500/250
		Magothy	105-690	500/13.9
N260	65	unsaturated upper glacial	0-65	2/ 1800/180
		upper glacial	65-90	1800/180
		Magothy	90-700	500/13.9
N296	75	unsaturated upper glacial	0-75	2/ 2500/250
		upper glacial	75-85	2500/250
		Magothy	85-670	500/13.9
N312	45	unsaturated upper glacial	0-45	2/ 2500/250
		upper glacial	45-100	2500/250
		Magothy	100-680	600/16.7
N334	70	unsaturated upper glacial	0-65	2/ 2500/250
		unsaturated Magothy	65-70	2/ 600/16.7
		Magothy	70-640	600/16.7
N358	75	unsaturated upper glacial	0-70	2/ 2500/250
		unsaturated Magothy	70-75	2/ 600/16.7
		Magothy	75-660	600/16.7
N376	40	unsaturated upper glacial	0-30	2/ 2500/250
		unsaturated Magothy	30-40	500/13.9
		Magothy	40-680	500/13.9
N413	25	unsaturated upper glacial	0-25	2/ 2500/250
		upper glacial	25-75	2500/250
		Magothy	75-615	600/16.7
N419	50	unsaturated upper glacial	0-50	2/ 2500/250
		upper glacial	50-80	2500/250
		Magothy	80-620	500/13.9

1/ Water-table depths are rounded to nearest 5 ft for use in the local and regional models. Depths measured from basin floor.

2/ Approximate conductivity of the saturated deposits.

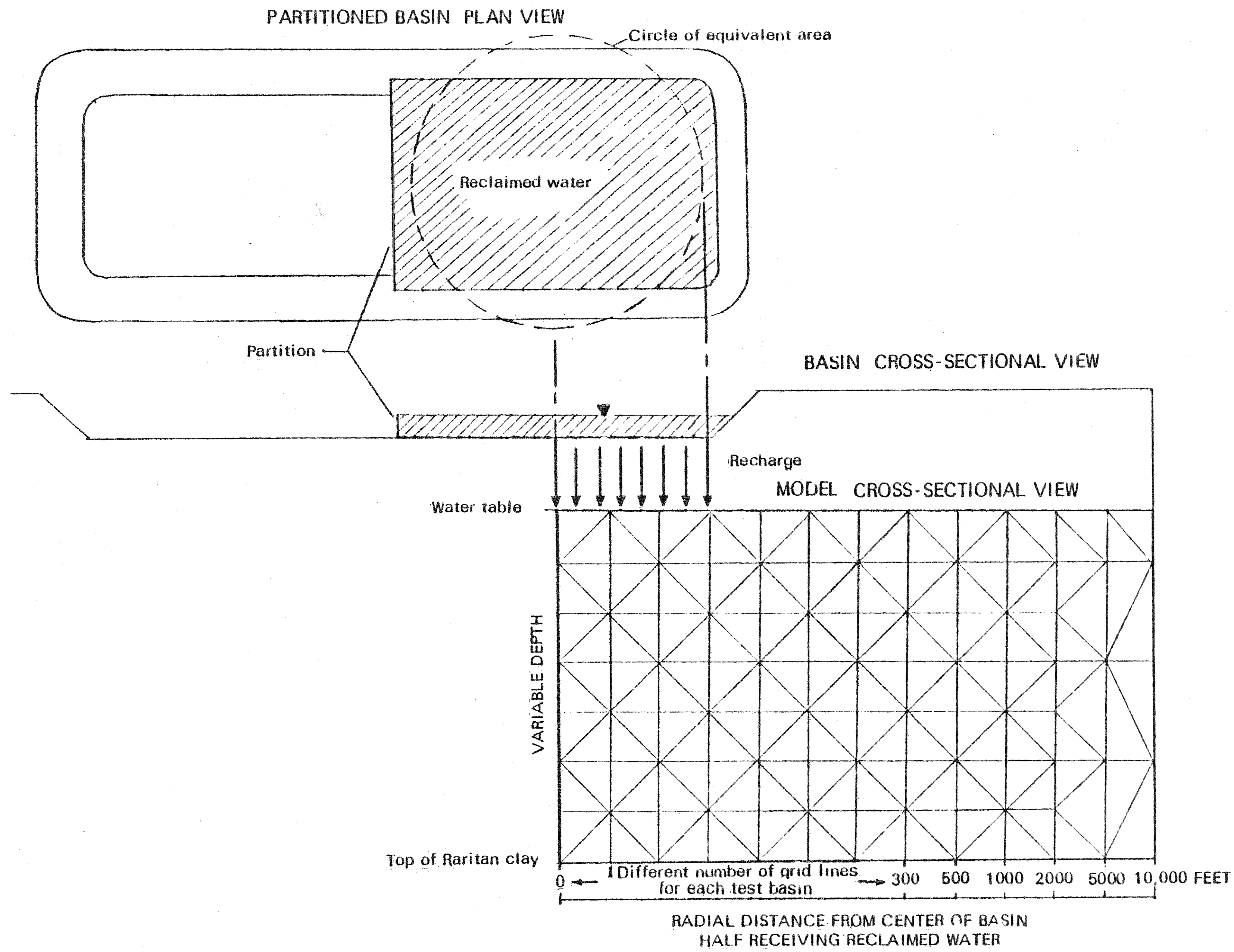

Figure 12.--Configuration of the Galerkin finite-element grid, showing location of grid components with respect to ponded area of a partitioned basin.

Regional Model

An electric-analog model of the ground-water flow system of Long Island was used to evaluate the regional effects of basin recharge within and about the study area. An electric-analog model is based on the analogy between the flow of water through a medium and the flow of electricity through a conductor. The values of hydrologic characteristics are converted to their electrical equivalents by constants and are incorporated into the model. The result is a simulation of ground-water flow in the aquifer system. The model aquifer system is divided into blocks, and the aquifer properties represented by each block are an average for the area represented. Model stresses and responses are also averaged for each block and are applied at the center of each block, which is termed a node. Nodes are the only points where stresses may be applied and water levels measured; stresses or water levels between nodes must be interpolated.

The Long Island analog model has five layers that together represent the Magothy, upper glacial, and Jameco aquifers together with any confining beds between them. (See table 1.) Thickness of the model layers varies with aquifer thickness, but horizontally, the nodes are uniformly spaced at intervals representing 6,000 feet. This representation of large areas gives the model its regional character. A more complete description of the Long Island analog model is given in Getzen (1977).

Long Island streams are represented on the regional model because approximately 90 percent of their flow originates as water from the water-table (upper glacial) aquifer. Streams are represented by the top-layer model nodes at appropriate locations, and the model shows the changes in streamflow that result from changes in water-table levels. The numerous small Long Island streams cannot be modeled in detail because of the model's scale, and no attempt is made to model seasonal fluctuations in streamflow. The method now used to model streams differs from the method originally used by Getzen (1977); it is described in detail in Harbaugh and Getzen (1977).

To simulate recharge through basins, the top-layer model block that represents the location of each basin must first be found. The location of these blocks and the corresponding node points (at the centers of these blocks) are shown in figure 13. Recharge for each basin is applied to its corresponding node. Five of the blocks contain two basins each; in these cases, the stress applied to the node represents total recharge by the two basins. Stresses are applied at a constant rate until steady-state conditions are reached.

Model Superposition

Each of the models gives the change in ground-water levels that would result from recharge alone. No other stresses are modeled, although ground-water levels, volume of streamflow, and stream length at the start of recharge operations can significantly affect water-table changes during recharge.

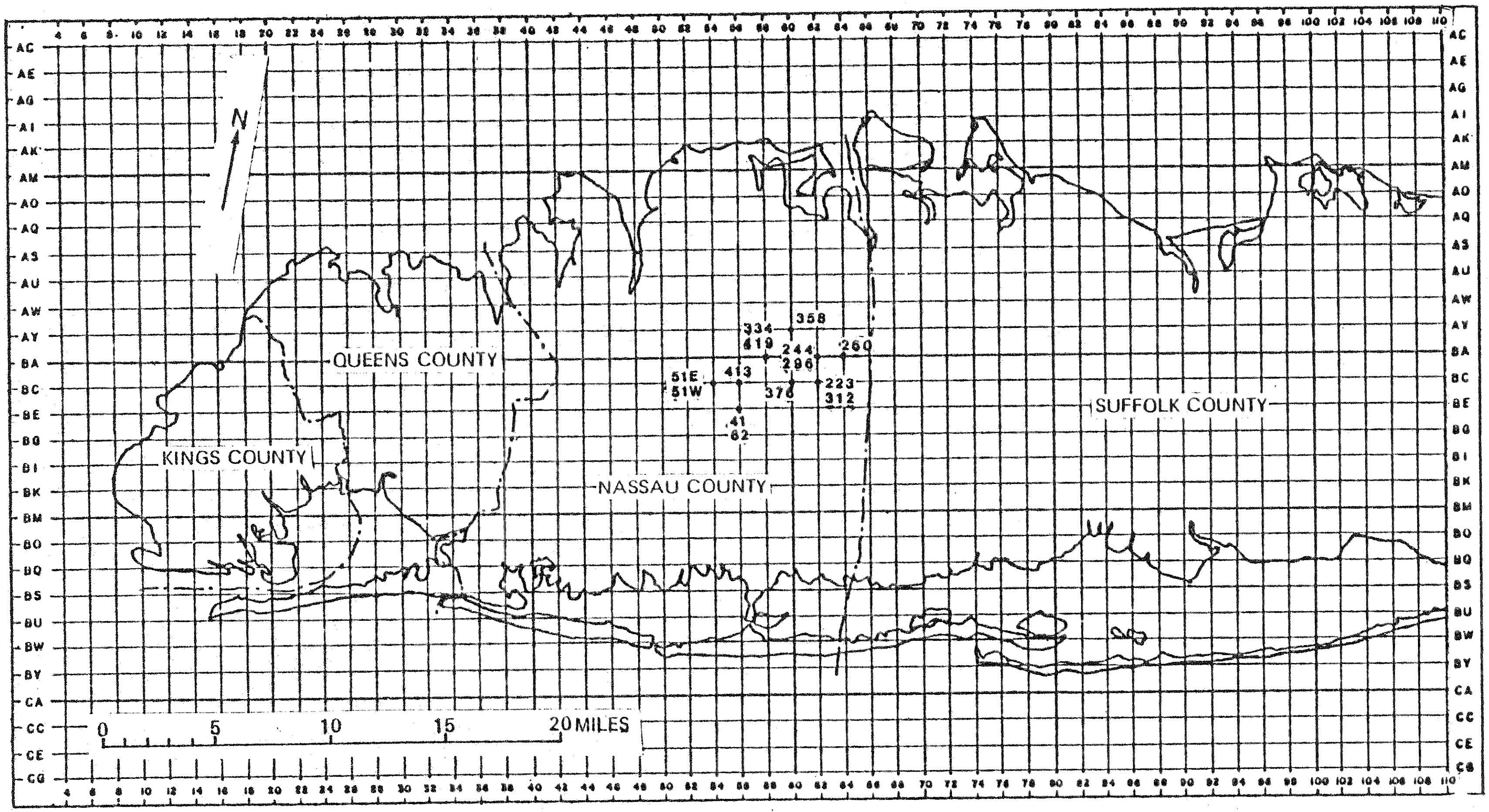

Figure 13.--Locations of the 14 test basins as represented by nodes in top layer of the Long Island analog model.

Distances between the floor of each test basin and the underlying water table were based on current (mid-1970's) water-table levels, and streamflows and stream lengths used in the regional model were mean values of 1943 to 1970.

The results from the local and regional models were combined using the principle of superposition, in which the solution of a problem involving two or more stresses together is equal to the sum of the solutions of each stress alone. Thus, the solution to a problem involving two wells could be solved by obtaining model results for each well, then adding the results. The superposition principle is somewhat inaccurate in basin simulation because water-level changes obtained either by the local or the regional model would change the transmissivity for the model water-table aquifer, which, in turn, would alter results in the other model. However, because both models assume a fixed transmissivity of the water-table aquifer under present conditions, superposition is considered reasonably valid. The effects of increased transmissivity were neglected; such an increase would result as the saturated thickness of the aquifer increased with water-table mounding. As a result, the simulated mounds are higher than if such transmissivity increases had been considered, and the predicted mound heights, therefore, are somewhat conservative.

Assuming that superposition is a valid approach, the mound at any one basin is the sum of (1) the mound produced by that basin alone, as determined by the digital model, and (2) the collective mound produced by the other 13 basins, as determined by the analog model. This approach would seem to require a separate regional simulation for each basin; however, as no basin has a major effect on the regional simulation, all regional simulations are approximately equal. Therefore, only one regional simulation was made, and it incorporated all basins.

Model Runs

It was necessary to determine how much reclaimed water could be added without raising the water table above the bottom of any of the basins, for this would preclude infiltration of storm runoff at the basin floor.1/ Ground-water levels in March 1974 (Koszalka, 1975) were used in computation of simulated levels. Storm-water infiltration through the 14 selected basins was assumed to be uniform during the modeled period; short-term deviations resulting from storm runoff into the basins were not incorporated into the model. (Accommodation of short-term deviations in volume of recharge caused by storm inflow is discussed in a later section.) Thus, results of the model runs show only the steady-state mound for the applied rate of infiltration after equilibrium is reached.

1/ As discussed in section "Model Results," a water-table mound extending to the floor of a basin from 1974 ground-water levels would permit additional infiltration of storm runoff because the regional water table is expected to decline as a result of sewering in Nassau County Sewage Disposal District 3 before large-scale basin recharge is begun.

The combined results from the local and regional models yield only the size of the mound caused by artificial recharge; the water level before recharge must be added to the height of the mound to give the final result. The maximum infiltration rate that could be used at each test basin without flooding it was found by trial and error. Several infiltration rates were calculated for each basin, and the results were checked to see if basin floors would be flooded. If water was calculated to rise above the bottom of one or more basins, infiltration rates were adjusted downward. If no flooding was predicted, infiltration rates were increased. The maximum infiltration rates that resulted from these runs are shown in table 8.

Because infiltration rate can be expected to decline during prolonged ponding, a second set of model runs was made with 50 $(gal/d)/ft^2$ or 0.28 ft/h) as the maximum infiltration rate. This rate is an estimate made in a water-supply study by Greely and Hansen (1971), as discussed earlier, and is termed the "proposed design rate of infiltration." Again, the trial-and-error method was used to reduce this infiltration rate for certain basins because some would still be flooded if the infiltration rate at all basins were 50 $(gal/d)/ft^2$ (0.28 ft/h). Rates resulting from this run are also shown in table 8.

Table 8.--Maximum and proposed design rates of infiltration, and resulting heights of water-table mounds at the 14 test basins

[Infiltration rates are in gallons per day per square foot]

Basin	Area of basin (ft^2)	Initial depth of water table below basin floor[1/]	Maximum infiltration rate: Infiltration rate	Maximum infiltration rate: Total mound height (ft below basin floor)	Maximum infiltration rate: Quantity of water added (Mgal/d)	Proposed design infiltration rate: Infiltration rate	Proposed design infiltration rate: Total mound height (ft below basin floor)	Proposed design infiltration rate: Quantity of water added (Mgal/d)
N41	21,800	30	90	30	2.0	50	20	1.1
N51 East	89,500	20	0	15	0	20	20	1.8
N51 West	143,000	20	0	15	0	15	20	2.1
N62	142,500	15	0	15	0	5	15	0.7
N223	38,100	45	75	40	2.8	50	25	1.9
N244	50,500	90	150	70	7.6	50	30	2.5
N260	53,500	65	100	55	5.4	50	30	2.7
N296	56,000	75	125	60	7.0	50	30	2.8
N312	35,400	45	150	40	5.3	50	20	1.8
N334	122,500	70	55	70	6.7	50	60	6.1
N358	124,000	75	60	75	7.4	50	60	6.2
N376	65,500	40	20	40	1.3	25	40	1.6
N413	89,500	25	30	25	2.7	45	25	4.0
N419	65,300	50	85	50	5.6	50	30	3.3
Totals					53.8			38.6

1/ Water-table depths were rounded to the nearest 5 ft for use in the digital and analog models.

Model Results

The change in the water-table altitude at each basin is shown in table 8 as the height of the mound resulting from recharge. The table summarizes values of the rate and quantity of water infiltrated at both the maximum-rate and proposed design-rate schemes. Figure 14 shows the predicted change in water-table altitude with proposed design infiltration rates of 50 (gal/d)/ft^2 (0.28 ft/h) or less; figure 15 shows the change in the water-table altitude with the maximum-rate scheme. These two figures were obtained by combining the results from local and regional models. Some judgment was used in the superposition and in combining the results of the two models; for example, the highest water-table contours

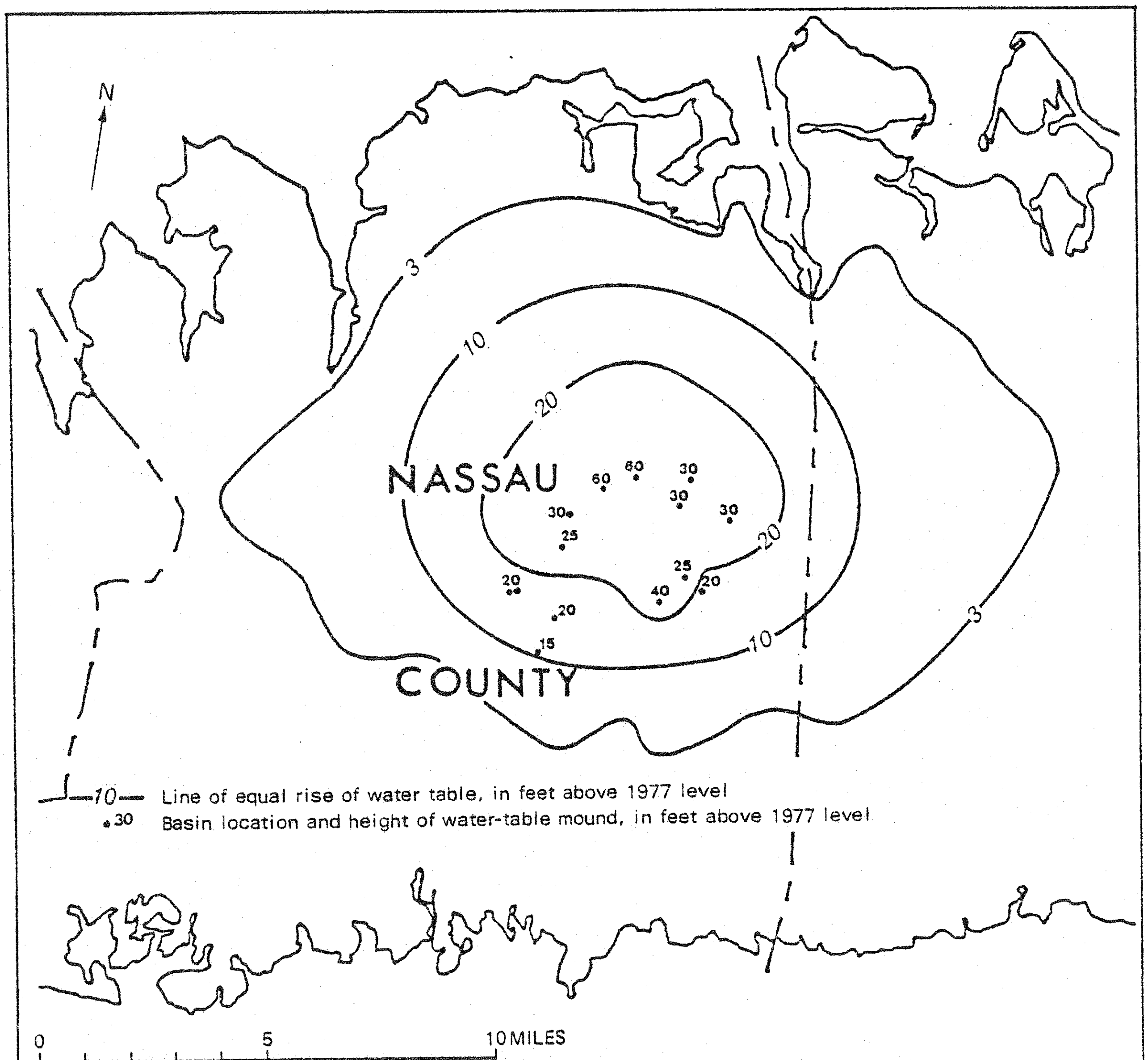

Figure 14.--Predicted buildup of water table in Nassau County as the result of artificial recharge at 14 test basins at proposed design infiltration rates.

in figures 14 and 15 are 20 ft and 40 ft, respectively. Although results of analog-model runs suggest that additional, higher water-table contours could be drawn within the mounds, they were not drawn for several reasons:

(1) the analog model has relatively low resolution and cannot accurately depict recharge at a point source;
(2) the analog model cannot yield accurate results under conditions of usually large stresses applied to very small areas;
(3) the analog model cannot accurately depict actual geologic conditions beneath each specific basin because of its regional nature; thus, inclusion of additional contours within the 40-ft and 20-ft contours of figures 13 and 14, respectively, would not be meaningful;

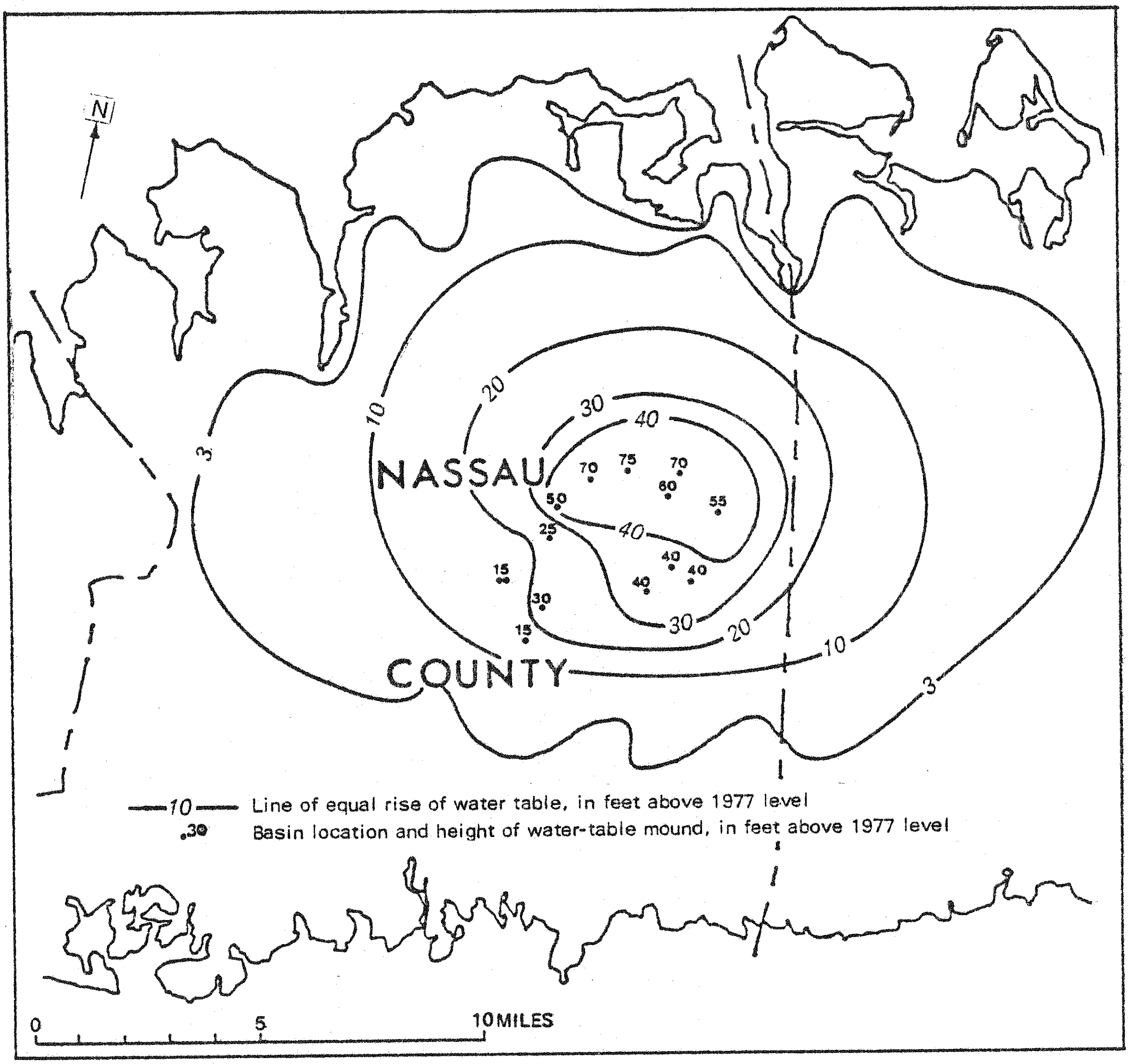

Figure 15.--Predicted buildup of water table in Nassau County as the result of artificial recharge at 14 test basins as maximum infiltration rates.

(4) As explained in the section "Model superposition," each basin was treated as a separate entity, and model results yield an accurate approximation of water-table mounding beneath each basin but a more regional value at some distance from the basin. Thus, the water-table configuration between basins that are near each other cannot be accurately defined by the method of superposition used in this study.

With few exceptions, proposed design infiltration rates are lower than maximum infiltration rates (table 8). Because the water table is close to land surface at basins 51 East, 51 West, and 62 (fig. 2), these basins cannot accommodate any reclaimed water when maximum rates of infiltration are applied at the other test basins. When the proposed design rates are applied, however, a reduction in the regional water-table buildup will allow these three basins to accommodate 4.7 Mgal/d of reclaimed water. In general, the volume of recharge through the 14 test basins at proposed design infiltration rates is about 70 percent of the volume attained at maximum infiltration rates.

The volumes of recharge attained at the proposed design infiltration rates of 50 (gal/d)/ft^2 (0.28 ft/h) or less are considerably smaller than the infiltration capacities of the basins. For example, considering only infiltration rate and infiltration area, basin 62 has an infiltration capacity of 25.6 Mgal/d with the partitioned-basin method of water application and an infiltration rate of 1.0 ft/h (a rate lower than that observed at the basin). Indeed, at an infiltration rate of 1.0 ft/h at each test basin, and with the infiltration areas of basin halves listed in table 8, the total theoretical infiltration capacity of the 14 test basins would be 197 Mgal/d.

The total volume of recharge possible at proposed design infiltration rates, however, is only 38.6 Mgal/d, or 20 percent of the theoretical infiltration capacity of the basins. The limiting factor in determining the quantity of reclaimed water that can be safely received by a basin, therefore, is the steady-state height of the underlying water table after artificial recharge, which, in turn, is dependent on the hydrogeologic character of the underlying deposits and the quantity of reclaimed water infiltrated at other recharge sites.

In 1975, streamflow in the modeled area of Long Island (fig. 16) averaged 194 Mgal/d. The steady-state increases in streamflow resulting from basin recharge at proposed design rates and maximum infiltration rates are given in table 9. Total streamflow in the modeled areas would increase by 12.5 percent and 17.9 percent at the proposed design rates and maximum infiltration rates, respectively. As shown in table 9, the largest projected increase in streamflow would be in area 1 because this area contains the 14 selected basins. Almost two-thirds of the water supplied to the basins becomes streamflow; the other third becomes subsurface outflow to Great South Bay and to Long Island Sound. The increase in streamflow resulting from artificial recharge through basins would partly offset the loss of streamflow predicted to result from sewering in Nassau County Sewage Disposal District 3 and the Suffolk County Southwest Sewer District (Kimmel and others, 1977). (The predicted water-table lowering is shown in fig. 17).

Table 9.--Increase in streamflow under steady-state conditions with proposed design and maximum infiltration rates

[Streamflow is in million gallons per day]

Modeled area[1/]	Proposed-design rate: Quantity of streamflow gain (Mgal/d)	Proposed-design rate: Percentage of total quantity recharged	Maximum rate: Quantity of streamflow gain (Mgal/d)	Maximum rate: Percentage of total quantity recharged
1	15.9	41	23.0	43
2	3.2	8	5.2	10
3	5.2	13	6.6	12
Totals	24.3	62	34.8	64

1/ Area 1 includes all of Nassau County south of the ground-water divide; area 2 includes southwest Suffolk County south of the ground-water divide; area 3 includes the north shore of Nassau and western Suffolk Counties north of the ground-water divide. Locations are shown in figure 16.

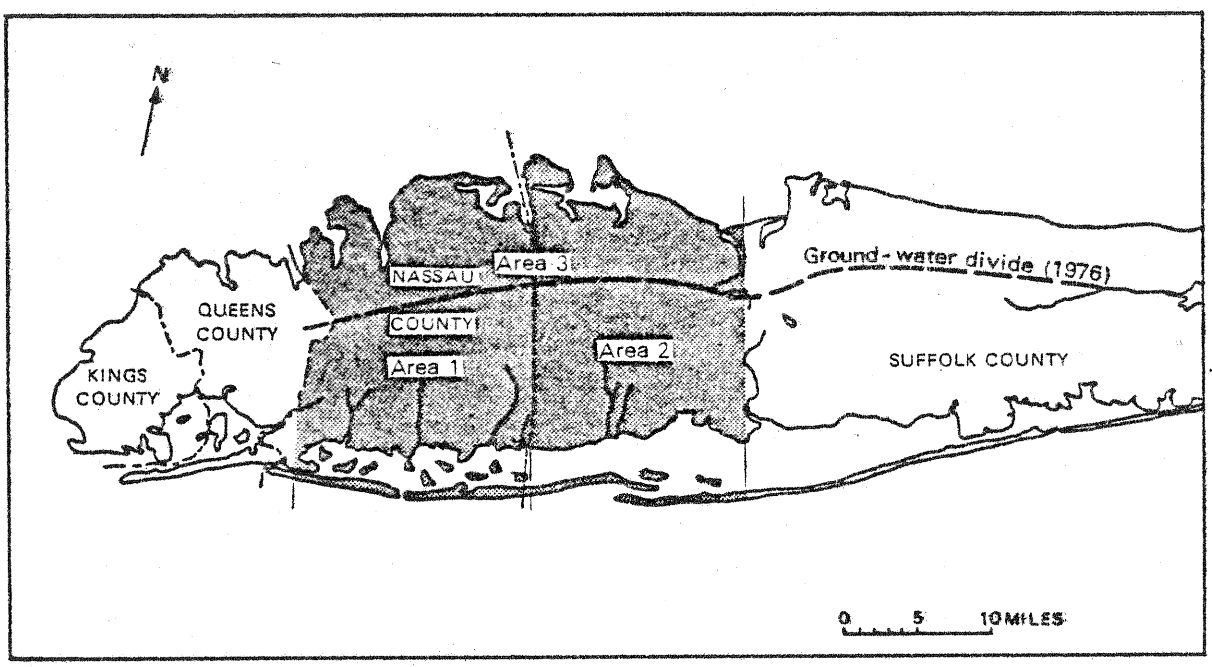

Figure 16.--Location of modeled stream area in Nassau and western Suffolk Counties, New York.

The changes in streamflow determined by the models are all based on the 1978 ground-water levels as initial conditions, and recharge is the only change applied. However, unless water levels are severely lowered as a result of large-scale sewering or other stresses, it is possible that artificial recharge will not be implemented. Kimmel and others (1977) analyzed water-table lowering caused by sewering; figure 17 shows changes in water levels that were predicted for Nassau County Sewage Disposal District 3 and Suffolk County Southwest Sewer District. Comparison of figure 17 with other data in this report shows that the net change resulting from recharge and sewering would leave the water table approximately 10 ft below the basin floor in the southernmost basins and between 10 and 20 ft below the basin floor in the central and northern basins. Because temporary buildup of the water table beneath basins during most storms on Long Island rarely exceeds 5 ft (Seaburn and Aronson, 1974), a water table lowered by sewering would considerably reduce the possibility that the water table would intersect the floor of a basin during infiltration of reclaimed water and storm runoff.

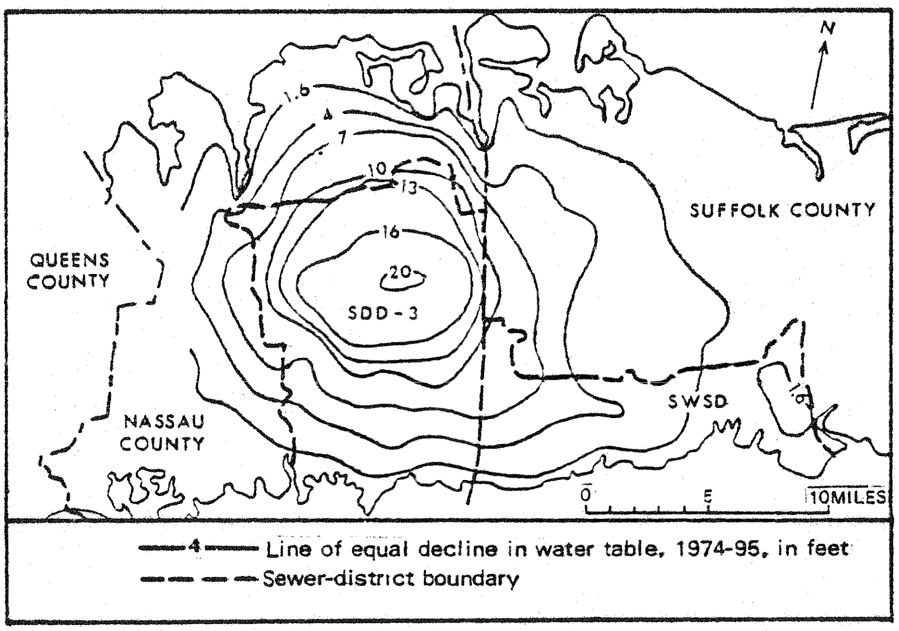

Figure 17.--Water-table decline in Nassau County and part of Suffolk County resulting from sewering in Sewage Disposal District 3 and Southwest Sewer District. [Modified from Kimmel and others, 1977.]

SUMMARY AND CONCLUSIONS

A survey of 205 storm-water basins south of the ground-water divide and north of Hempstead Turnpike in Nassau County, Long Island, N.Y. was made to determine which basins would be best suited for infiltration of reclaimed water. Of the 50 largest basins, 14 were selected for detailed study of operating characteristics and the effects of artificial recharge on the local and regional water table.

The total maximum infiltration area of the 14 selected basins is 60.2 acres or 2,620,900 ft^2. If a 5-ft high partition were constructed to divide each basin approximately in half, and if reclaimed water were applied in half of each basin to a depth of 5 ft, a total of 25.2 acres or 1,097,100 ft^2 would be available for supplemental recharge; the remaining infiltration area would be used for disposal of storm runoff. Basin halves would be operated on an application-and-rest cycle, whereby the function of each half would be periodically reversed to permit restoration of infiltration capacity.

Mathematical-, analog-, and digital-model analyses assessed (1) the ability of the 14 test basins to accommodate large volumes of reclaimed water and storm runoff simultaneously, and (2) the ability of the deposits underlying the basins to transmit and store reclaimed water without excessive water-table buildup. The results of these analyses are summarized in the following paragraphs.

(1) All 14 selected basins could accommodate reclaimed water and runoff from large-magnitude storms without overflowing, depending on storm duration and intensity, infiltration rate, runoff coefficient, and partition height.

(2) Storm durations that produce maximum loading (stress) at the test basins range from 4 to 14 hours, depending on storm magnitude. Differences between durations of the most stressful storms at each basin reflect differences in volume of runoff received by each basin and volume/area relations of the basins.

(3) The duration of storm-runoff containment is a limitation on the use of storm-water basins for artificial recharge with reclaimed water. If a large-magnitude storm should occur before a basin had disposed of runoff from a previous storm, it might overflow.

(4) Basin 62 has the largest reserve capacity for storing storm runoff and reclaimed water; only 36 to 55 percent of its available storage capacity would be used during 10- to 100-year storms, respectively. Basin 41 has the smallest reserve capacity; 89 to 134 percent of its capacity would be required for stored water during 10- to 100-year storms, respectively.

(5) Basins 62, 296, 334, and 376 would not overflow during large-magnitude storms with simultaneous infiltration of reclaimed water. The remaining eight basins would overflow during the most stressful 25-, 50-, or 100-year storms. None of the basins would overflow during a 10-year storm.

(6) Of those basins subject to overflow during artificial recharge, basin 41 would have the largest percentage of overflow compared to volume of total runoff, which suggests that it is the least efficient.

(7) Duration of runoff storage is shortest at basin 51 West, where it ranges from 29 to 30 hours, depending on storm magnitude, and is longest at basin 413, where it ranges from 60 to 66 hours. Even the least efficient of the test basins will dispose of runoff from the most severe 100-year storms within 3 days, if infiltration rates above and below the top of the partition do not fall below those assumed in the analyses.

(8) The infiltration rate below the top of a 5-ft partition could decline to zero at all but basin 260 during a 10-year storm without overflow. Basins 62, 296, 334, and 376 would not overflow even during 100-year storms if the infiltration rate below the top of the partition declined to zero. Basins 41, 223, and 260 would require infiltration rates of more than 1.0 ft/h below the top of the partition to prevent overflow during 100-year storms.

(9) Runoff coefficients for certain of the test basins could be considerably higher than the 40-percent design coefficient without causing overflow. For example, the runoff coefficient at basin 62 could be as high as 94 percent during a 10-year storm or 66 percent during a 100-year storm without producing overflow. In contrast, maximum allowable runoff coefficients at basin 41 range from 46 to 28 percent during 10- to 100-year storms, respectively.

(10) A runoff coefficient of about 25 percent was determined for basin 41, 30 percent for basin 51 West, 25 percent for basin 62, 30 percent for basin 296, 20 percent for basin 312, and 20 percent for basin 358 during an actual storm. These values are all smaller than the maximum allowable coefficients required to prevent overflow at these basins. The other eight test basins would probably not overflow as long as runoff coefficients in effect during a storm were not extraordinarily high.

(11) The tendency of certain basins to overflow could be significantly reduced by (a) enlarging the basin to accommodate excess runoff; (b) lowering the height of the dividing partition; (c) diverting excess runoff to another basin; or (d) temporarily halting inflow of reclaimed water so that additional infiltration area would become available for storm runoff.

(12) Partition height governs the duration of the application-and-rest cycle of reclaimed water in a partitioned basin. As infiltration rate declines during ponding, the time required for the level of reclaimed water to rise to the top of the partition depends on partition height. Basins that would overflow during certain large-magnitude storms might not do so if partition height and, hence, application-and-rest cycle, were reduced.

(13) The analyses of basin performance were based on storm durations that produce the greatest stress (loading) on the basins. Under real conditions, worst-storm conditions are unlikely, and, accordingly, results of basin performance analyses are probably conservative.

(14) The amount of reclaimed water that can be disposed of by the 14 test basins without flooding the basin floor is 53.8 Mgal/d at maximum rates of recharge and 38.6 Mgal/d at proposed design rates of recharge (< 0.28 ft/h). In general, the volume of water at proposed design recharge rates at the 14 test basins is about 70 percent of the volume at maximum recharge rates.

(15) Basins 51 east, 51 west, and 62 cannot accept reclaimed water when recharge rates are at a maximum at the other 11 test basins because the regional water-table rise would flood them. When proposed design rates of recharge are used, however, these three basins could accept 4.7 Mgal/d of reclaimed water.

(16) Proposed design recharge rates permit infiltration of considerably less water volume than the infiltration capacities of the basins. The total volume of water for the 14 test basins at proposed design recharge rates is 38.6 Mgal/d, or 20 percent of the basins' theoretical maximum infiltration capacity.

(17) The regional water-table rise that would result from artificial recharge at the 14 test basins would partly offset the water-table decline that will result from sewering in parts of Nassau and Suffolk Counties.

(18) The principal limiting factor in the quantity of reclaimed water that can be accommodated by a basin is the height of the underlying water table, which is temporarily raised during artificial recharge.

(19) During supplemental recharge, total streamflow in the modeled area would increase by 12.5 percent and 17.9 percent, at proposed design and maximum rates of recharge, respectively. Almost two-thirds of the added water would leave the system as streamflow; the other third would leave as subsea outflow to the Great South Bay and Long Island Sound. The increase in streamflow would be partly offset by the decrease in streamflow that is predicted to result from sewering in parts of Nassau and Suffolk Counties.

The study has shown that the 14 selected basins can transmit large quantities of reclaimed water to the ground-water reservoir with little likelihood of overflow or excessive water-table buildup, under proper conditions. The use of present basins for such recharge would be advantageous because of high land costs and restricted land availability in the areas most in need of artificial recharge. However, the 14 selected basins can counteract only 42 percent of the 92-Mgal/d water deficiency anticipated for Nassau County in the year 1990. To accommodate the total volume of water needed, other available basins would have to be incorporated into the recharge system, but, because the other basins in Nassau County are relatively small, new, large basins would be needed, and additional studies that include mathematical-, digital-, and analog-model analyses would be required to assess the hydrologic effects of large-scale recharge in Nassau County.

REFERENCES CITED

Aronson, D. A., 1976a, Preliminary selection of storm-water basins suitable for infiltration of reclaimed water in Nassau County, Long Island, New York: U.S. Geological Survey Open-File Report 76-668, 39 p.

_________, 1976b, Evaluation of alternative methods of supplemental recharge by storm-water basins on Long Island, New York: U.S. Geological Survey Open-File Report 76-470, 56 p.

_________, 1977, Determination of runoff coefficients of storm-water-basin drainage areas on Long Island, New York by using maximum-stage gages: U.S. Geological Survey Journal of Research, vol. 6, no. 1, p. 11-21.

Aronson, D. A., and Prill, R. C., 1977, Analysis of the recharge potential of storm-water basins on Long Island, New York: U.S. Geological Survey Journal of Research, vol. 5, no. 3, p. 307-318.

Chow, V. T., 1964, Runoff, in Chow, V. T., ed., Handbook of applied hydrology: New York, McGraw-Hill Book Company, Section 8, p. 22.

Cohen, Philip, Franke, O. L., and Foxworthy, B. L., 1968, An atlas of Long Island's water resources: New York State Water Resources Commission Bulletin 62, 117 p.

Desai, C. S., and Abel, J. F., 1972, Introduction to the finite element method: New York, Van Nostrand Reinhold Company, 477 p.

Franke, O. L., and Cohen, Philip, 1972, Regional rates of ground-water movement on Long Island, New York, in Geological Survey Research, 1972: U.S. Geological Survey Professional Paper 800-C, p. C271-C277.

Getzen, R. T., 1977, Analog-model analysis of regional three-dimensional flow in the ground-water reservoir of Long Island, New York: U.S. Geological Survey Professional Paper 982, 49 p.

Greeley and Hansen, Engineers, 1971, Report: Comprehensive public water supply study, Nassau County, New York: Chicago, Illinois, CPWS-60, 205 p.

Harbaugh, A. W., and Getzen, R. T., 1977, Stream simulation in analog model of the ground-water system on Long Island, New York: U.S. Geological Survey Water-Resources Investigation 77-58, 15 p.

Kimmel, G. E., Ku, H. F. H., Harbaugh, A. W., Sulam, D. J., and Getzen, R. T., 1977, Analog model prediction of the hydrologic effects of sanitary sewerage in southeast Nassau and southwest Suffolk Counties, New York: Long Island Water Resources Bulletin LIWR-6, 25 p.

Koszalka, E. J., 1975, The water table on Long Island, in March 1974: Long Island Water Resources Bulletin LIWR-5, 7 p., 3 maps.

Ku, H. F. H., Vecchioli, John, and Cerrillo, L. A., 1975, Hydrogeology along the proposed barrier-recharge-well alinement in southern Nassau County, Long Island, New York: U.S. Geological Survey Hydrologic Investigations Atlas HA-502, 1 sheet.

McClymonds, N. E., and Franke, O. L., 1972, Water-transmitting properties of Long Island's aquifers: U.S. Geological Survey Professional Paper 627-E, 24 p.

McGauhey, P. H., and Krone, R. B., 1967, Soil mantle as a wastewater treatment system: University of California, Berkeley, SERL report 67-11, 201 p.

Miller, J. F., and Frederick, R. H., 1969, The precipitation regime of Long Island, New York: U.S. Geological Survey Professional Paper 627-A, 21 p.

Perlmutter, N. M., and Geraghty, J. J., 1963, Geology and ground-water conditions in southern Nassau and southeastern Queens Counties, Long Island, New York: U.S. Geological Survey Water-Supply Paper 1616-A, 205 p.

Pinder, G. F., and Gray, W. G., 1977, The finite element method in surface and subsurface hydrology· New York, Academic Press, 310 p.

Pluhowski, E. J., and Kantrowitz, I. H., 1964, Hydrology of the Babylon-Islip area, Suffolk County, Long Island, New York: U.S. Geological Survey Water-Supply Paper 1768, 119 p.

Prill, R. C., and Aronson, D. A., 1978, A ponding-test procedure for assessing the infiltration capacity of storm-runoff basins: U.S. Geological Survey Water-Supply Paper 2049, 29 p.

Seaburn, G. E., 1970, Preliminary results of hydrologic studies at two recharge basins on Long Island, New York: U.S. Geological Survey Professional Paper 629-C, 17 p.

Seaburn, G. E., and Aronson, D. A., 1974, Influence of recharge basins on the hydrology of Nassau and Suffolk Counties, Long Island, New York: U.S. Geological Survey Water-Supply Paper 2031, 66 p.

Zienkiewicz, O. C., 1971, The finite element method in engineering science: London, McGraw-Hill, 521 p.